APPENDIX C

Geotechnical Exploration

Geotechnical Exploration

401 Alberto Way
Los Gatos, California

Submitted to:
Mr. Shane Arters
LP Acquisitions, LLC
% Lamb Partners, LLC
525 Middlefield Road, Suite 118
Menlo Park, CA 94025

Prepared by:
ENGEO Incorporated

July 17, 2015
Revised August 13, 2015

Project No:
12175.000.000

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Mr. Shane Arters
LP Acquisitions, LLC
℅ Lamb Partners, LLC
525 Middlefield Road, Suite 118
Menlo Park, CA 94025

Subject: 401 Alberto Way
Los Gatos, California

GEOTECHNICAL EXPLORATION

Dear Mr. Arters:

As requested, we completed this geotechnical exploration for your proposed office building project in Los Gatos, California. The accompanying report presents our field exploration and laboratory testing with our conclusions and recommendations regarding the proposed development at the site.

Our findings indicate that the site is suitable for the proposed development provided the recommendations and guidelines provided in this report are implemented during project planning, design and construction. We are pleased to have been of service to you on this project and are prepared to consult further with you and your design team as the project progresses.

Sincerely,

ENGEO Incorporated

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this geotechnical report, as described in our proposal dated June 16, 2015, is to provide design-level geotechnical recommendations associated with the proposed office building development of the site.

We performed the following services:

- Review of available literature, previous reports and geologic maps for the study area.
- Subsurface exploration consisting of three soil borings.
- Laboratory testing of materials sampled during the field exploration.
- Geotechnical data analyses.
- Report preparation summarizing our conclusions and recommendations for the proposed development.

Our services are based on the following plan set:

- A Planning Application for 401-409 Alberto Way, Los Gatos, prepared by Architectural Technologies and dated May 15, 2015.

We prepared this report exclusively for LP Acquisitions, LLC and their design team consultants. ENGEO should review any changes made in the character, design or layout of the development to modify the conclusions and recommendations contained in this report, as necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without the express written consent of ENGEO.

1.2 SITE LOCATION AND DESCRIPTION

The roughly 2.15-acre property is located at 401 Alberto Way in Los Gatos, California. The site is generally bounded by residential development to the north, Los Gatos Saratoga Road to the south, Highway 17 to the west, and Alberto Way to the east (Figures 1 and 2). Based on a recent site visit, the project area is currently occupied by three 2-story office buildings with associated at-grade parking and landscape areas.

1.3 PROPOSED DEVELOPMENT

Based on the referenced plan prepared by Architectural Technologies (dated May 15, 2015), we anticipate the new development will consist of a podium structure including two 2-story office buildings encompassing areas of 47,800 square feet (Building 1) and 45,000 square feet (Building 2) over a two-level below-grade parking garage. The parking garage is shown to underlie the entirety of Building 1 and the majority of Building 2, with the exception of the southern portion of Building 2. Associated improvements include an at-grade parking area, trash enclosure, and landscaped areas. Based on conversations with you, it is our understanding that the office buildings will consist of steel-framed construction.

1.4 AERIAL PHOTOGRAPH REVIEW

We reviewed individual aerial photographs of the site dated 1939, 1948, 1950, 1956, 1968, 1974, 1982, 1993, 1998, 2005, 2006, 2009, 2010 and 2012 provided by Environmental Data Resources (EDR).

The site appears to be vacant land with some vegetation and agricultural use until the time of the photograph dated 1968, at which time three structures with associated paved parking areas are first visible within the site. The site resembles present-day conditions throughout the remaining photographs reviewed.

Additionally, we reviewed numerous stereo-paired images (dated 1937 through 2005) to investigate potential geologic hazards impacting the subject site. Observations made from examining the stereo-paired images were utilized in our geologic review and are discussed in the appropriate sections below.

1.5 PREVIOUS GEOTECHNICAL STUDIES ON NEIGHBORING PROPERTIES

1.5.1 55 Los Gatos Saratoga Road, Earth Systems Geotechnologies (ESG)

55 Los Gatos Saratoga Road, which is located immediately east of the subject site on the opposite side of Alberto Way, was explored by ESG in 2008 for a proposed office building and parking lot. ESG's subsurface exploration consisted of advancing two borings to depths of approximately 41 and 19½ feet below the ground surface (bgs). The borings generally encountered very dense sands and gravels with varying clay content to a depth of approximately 33½ feet bgs, below which depth shale bedrock was observed. Groundwater was encountered by ESG at depths ranging between approximately 18½ and 21 feet bgs. These subsurface findings were utilized in our review of geologic hazards, as discussed in the sections below.

2.0 GEOLOGIC CONDITIONS

2.1 REGIONAL GEOLOGY

Regional geologic mapping by McLaughlin et al. (2000, Figure 3) identifies Holocene-age alluvial fan deposits (Qhf) underlying the site. Similarly, regional mapping by Dibblee (2005) indicates the site is underlain by Quaternary-age sand and gravel of major stream channels (Qg), presumably deposited by nearby Los Gatos Creek.

2.2 REGIONAL FAULTING AND SEISMICITY

Regional geologic mapping by McLaughlin et al. (2001) depicts a concealed splay of the Berrocal fault approximately 200 feet to the south of the site, trending in a direction roughly parallel to Los Gatos Saratoga Road. Similarly, the Fault Lineament & Coseismic Deformation Map for the Town of Los Gatos General Plan Update (Nolan Associates, 1999) depicts the same concealed splay approximately 250 to 300 feet south of the site.

The site is not located within a State of California Earthquake Fault Hazard Zone (Los Gatos Quadrangle, 1991) for active faults, and no known faults cross the site. However, the southern two-thirds of site is located within a Santa Clara County Fault Rupture Hazard Zone (2012) due to the nearby mapped trace of the Berrocal fault to the south of the site, which is identified as a Quaternary-age fault by the USGS (USGS, Quaternary Fault and Fold Database). Additionally, the site is located within a zone for high fault rupture hazard potential as depicted on the Fault Rupture Hazard Zones Map for the Town of Los Gatos General Plan Update (Nolan Associates, 1999). Review of the Fault Lineament & Coseismic Deformation Map for the Town of Los Gatos General Plan Update (Nolan Associates, 1999) and Plate 1 of the USGS Open File Report 95-820 (Schmidt et al., 1995) indicates that the site is not located in an area that experienced a concentration of coseismic ground deformation or damage to the ground surface as a result of the 1989 Loma Prieta Earthquake.

Nearby active[1] and potentially active faults include the Berrocal fault, located approximately 200 to 300 feet south and 0.3 mile north of the site; Monte Vista-Shannon fault located approximately 1.4 miles north of the site; and the San Andreas fault, located approximately 3.4 miles southwest of the site.

Because of the presence of nearby active faults, the Bay Area Region is considered seismically active. Numerous small earthquakes occur every year in the region, and large (>M7) earthquakes have been recorded and can be expected to occur in the future. Figure 4 shows the approximate locations of these faults and significant historic earthquakes recorded within the Greater Bay Area Region.

3.0 FIELD EXPLORATION

3.1 EXPLORATORY BORINGS

The field exploration for this study included advancing three exploratory borings within the project site on June 27, 2015. The borings were drilled to depths ranging from approximately 15 feet bgs to 40½ feet bgs using a track-mounted rig equipped with either 8-inch-diameter hollow-stem augers or 6-inch-diameter solid flight augers. Figure 2 presents the approximate locations of the exploratory borings obtained by taping or pacing from existing features. As a

[1] An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,000 years) (Hart, 1997).

result, the mapped locations should be considered only as accurate as the methods used to determine them.

The borings were logged in the field and soil samples were collected using either a 2½-inch inside diameter (I.D.) California-type split-spoon sampler fitted with 6-inch-long brass liners or a 2-inch outside diameter (O.D.) Standard Penetration Test split-spoon sampler. The penetration of the samplers into the native materials was recorded as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring logs record blow count results as the actual number of blows required for the last 1 foot of penetration; no conversion factors have been applied. The samplers were driven with a 140-pound hammer falling a distance of 30 inches employing an automatic hammer system. The field logs were then used to develop the report boring logs, which are presented in Appendix A.

The boring logs depict subsurface conditions within the borings at the time of the exploration. Subsurface conditions at other locations may differ from conditions occurring at these boring locations, and the passage of time may result in altered subsurface conditions. In addition, stratification lines represent the approximate boundaries between soil types, and the transitions may be gradual.

Upon completion, the test holes were backfilled with grout.

3.2 LABORATORY TESTING

We performed the following laboratory tests on select samples recovered during boring operations:

TABLE 3.2-1
Laboratory Testing

Soil Test	Testing Method	Location of Results
Natural Unit Weight and Moisture Content	ASTM D7263	Appendix A
Atterberg Limits	ASTM D4318	Appendix B
Grain Size Distribution	ASTM D422	Appendix B
Unconfined Compression	ASTM D2166	Appendix B
Unconsolidated Undrained Triaxial	ASTM D2850	Appendix B
Corrosivity Testing (Redox, pH, Resistivity, Chloride, Sulfate)	ASTM D-1498, D-4972, G57, D-4658M, D-4327	Appendix C

The laboratory test results are shown on the borelogs (Appendix A), with individual test results presented in Appendices B and C.

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3.3 SUBSURFACE CONDITIONS

In general, our exploratory borings encountered medium dense to dense clayey sands to depths ranging between 10 to 21 feet bgs, which in turn were underlain by medium dense to very dense clayey gravels to depths of approximately 29 to 33 feet bgs. Bedrock consisting of a weak, closely fractured shale was encountered below the gravelly soils. Similar soils and depth to bedrock was observed by ESG on the neighboring property to the east at 55 Los Gatos Saratoga Road.

3.4 GROUNDWATER

Groundwater was encountered during our subsurface exploration and during the exploration by ESG on the neighboring property to the east at depths of approximately 18½ to 21 feet bgs. Plate 1.2 of the Seismic Hazard Zone Report for the Los Gatos Quadrangle (2002) indicates historic groundwater highs between approximately 10 to 20 feet below the ground surface.

Fluctuations in groundwater levels should be expected during seasonal changes or over a period of years because of precipitation changes, perched zones, changes in drainage patterns, and irrigation.

4.0 GEOLOGIC AND GEOTECHNICAL HAZARDS

The site was evaluated with respect to known geologic and other hazards common to the area. The primary hazards and the risks associated with these hazards with respect to the planned development are discussed in the following sections of this report.

4.1 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, soil liquefaction, and lateral spreading. These hazards are discussed in the following sections. Based on topographic data, risk from tsunamis or seiches is considered low to negligible at the site.

4.1.1 Ground Rupture

As described above, the site is not located within a State of California Earthquake Fault Hazard Zone (Los Gatos Quadrangle, 1991) and no known faults cross the site. Review of aerial images provided by EDR and stereo-paired images did not reveal any visible lineaments in the vicinity or crossing the subject site. Review of the Fault Lineament & Coseismic Deformation Map for the Town of Los Gatos General Plan Update (Nolan Associates, 1999) and Plate 1 of the USGS Open File Report 95-820 (Schmidt et al., 1995) indicates that the site is not located in an area that experienced a concentration of coseismic ground deformation or damage to the ground surface as a result of the 1989 Loma Prieta Earthquake.

However, the site is located within a Santa Clara County Fault Rupture Hazard Zone and high fault rupture hazard potential zone (Nolan Associates, 1999) due to the nearby mapped trace of the Berrocal fault, located approximately 200 to 300 feet south of the site. The Berrocal fault, which trends in an east-west direction in the project area, is a southwest-dipping, reverse dextral-oblique fault zone (USGS Quaternary Fault and Fold Database). Should the fault zone pass through the subject site, a significant vertical offset of the geologic contact between bedrock and overlying sediments would be expected across the northern and southern portions of the site. However, Borings 1-B2 and 1-B3, advanced roughly 300 feet apart on the southern and northern sides of the site, respectively, encountered shale bedrock at approximately the same elevation (between approximately El. 307 and 309.5). Bedrock was encountered at a similar depth by ESG in a boring advanced at a neighboring property immediately east of the site.

Based on the absence of observable photo lineaments in the vicinity or crossing the site, consistently mapped location of the Berrocal fault to the south of the site, and lack of coseismic deformation observed at the site and in the immediate vicinity of the site following the 1989 Loma Prieta Earthquake, it is our opinion that ground rupture is unlikely at the subject property.

4.1.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the latest California Building Code (CBC) requirements, as a minimum.

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

4.1.3 Soil Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded fine sands below the groundwater table. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop and liquefaction of susceptible soil to occur.

Review of the Seismic Hazard Zones Map for the Los Gatos Quadrangle (2002) indicates that the site is located within a mapped liquefaction zone. To assess liquefaction potential, we performed liquefaction analyses on two exploratory borings (1-B2 and 1-B3) advanced at the site. We assumed a groundwater level 15 feet below the existing ground surface based on groundwater measurements made during our exploration and as reported in the referenced study prepared by ESG. Additionally, we utilized a PGA of 1.00g and a Mw of 8.0. Our analyses were based on guidelines provided in DMG Special Publication 117A (2008) and methods developed by Youd et al. (NCEER 1998) (2001), Seed (2003), and Boulanger and Idriss (2008).

Based on our analysis, soils encountered at Boring 1-B2 were identified as too dense to liquefy based on review of the blow counts. Soils encountered at Boring 1-B3 were identified as potentially liquefiable in accordance with methods developed by Seed (2003) and Boulanger and Idriss (2008) but were identified as too dense to liquefy based on methods developed by Youd et al. (NCEER 1998) (2001).

As previously mentioned, the majority of the site (with the exception of the southern portion of Building 2) will be excavated to an estimated depth of approximately 20 feet to accommodate the proposed subterranean parking garage. Based on the methodologies outlined above, it is our opinion that the gravel deposits at portions of the site below a depth of approximately 20 feet (with a cumulative thickness of roughly 9 feet) are potentially liquefiable. Additionally, for portions of the site not within the proposed subterranean parking garage, it is our opinion that gravel deposits at portions of the site below a depth of approximately 15 feet (with a cumulative thickness of roughly 14 feet) are also potentially liquefiable. Liquefaction calculations are included in Appendix D.

4.1.4 Liquefaction-Induced Ground Settlement

Our liquefaction analyses indicate that gravel deposits up to 9 feet thick below the bottom of the proposed parking garage (estimated to be 20 feet below grade) may potentially liquefy and result in vertical settlements of approximately 1 inch. Additionally, our liquefaction analyses indicate that gravel deposits up to 14 feet thick for portions of the site not within the proposed parking garage may potentially liquefy and result in vertical settlements of approximately 2 inches.

4.1.5 Lateral Spreading

Lateral spreading can occur in weaker soils on slopes and adjacent to open channels that are subject to strong ground shaking during earthquakes. Based on the relatively flat site topography, variability in density of coarse-grained deposits, and the location of the nearest drainage channel (Los Gatos Creek) roughly 500 feet to the west, it is our opinion that there is a low potential for liquefaction-induced lateral spreading.

4.2 EXISTING FILL

The site is currently occupied by three structures and associated improvements. As such, buried foundation elements and underground utilities may be present on the site.

Existing fills could undergo vertical movement that is not easily characterized and could ultimately be inadequate to effectively support the proposed building loads. In general, undocumented fills should be excavated, and if deemed suitable for reuse, replaced as engineered soil fill. Due to the proposed subterranean parking garage, it is our opinion that the majority of existing fills (if present) will be removed as a result of the garage excavation. However, the extent and quality of existing fills should be evaluated and mitigated during grading activities.

4.3 EXPANSIVE SOILS

Expansive soils shrink and swell as a result of moisture changes. This can cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Atterberg Limits testing performed on samples collected during our field exploration yielded Plasticity Indices (PI) of 19 and 21, indicating a moderate expansive potential of onsite soils.

Successful construction on expansive soils requires special attention during grading. It is imperative to keep exposed soils moist by occasional sprinkling. If the soils dry, it is extremely difficult to remoisturize the soils (because of their clayey nature) without excavation, moisture conditioning, and recompaction.

Conventional grading operations, incorporating fill placement specifications tailored to the expansive characteristics of the soil, and use of a mat foundation are common, generally cost-effective measures to address the expansive potential of the foundation soils. Based upon our initial findings, the effects of expansive soils are expected to pose a low impact when mitigated.

4.4 GROUNDWATER

Groundwater was encountered during our subsurface exploration and during the exploration by ESG on the neighboring property to the east at depths of approximately 18½ to 21 feet bgs. Plate 1.2 of the Seismic Hazard Zone Report for the Los Gatos Quadrangle (2002) indicates historic groundwater highs between approximately 10 to 20 feet below the ground surface.

Based on the above, we recommend using a design groundwater level of 12 feet below existing grade. Fluctuations in groundwater levels should be expected during seasonal changes or over a period of years because of precipitation changes, perched zones, changes in drainage patterns, and irrigation.

4.5 FLOODING

The project Civil Engineer should be consulted on the potential for localized flooding at the subject site. The review should also include a determination of whether the site falls below the 100-year flood plain elevation.

4.6 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS

Considering nearby faults, we provide the 2013 CBC seismic parameters for your use in foundation design. The seismic design parameters presented in the 2013 CBC are based upon the 2012 International Building Code and the ASCE standard "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10) published in 2010. To obtain 2013 CBC seismic parameters, we used the USGS Seismic Design Map online tool to develop ASCE 7-10 seismic design parameters.

TABLE 4.6-1
2013 CBC Seismic Design Parameters

Parameter	Design Value
Site Class	C
Mapped MCE_R spectral response accelerations for short periods, S_S (g)	2.66
Mapped MCE_R spectral response accelerations for 1-second periods, S_1 (g)	1.01
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.30
MCE spectral response accelerations for short periods, S_{MS} (g)	2.66
MCE spectral response accelerations for 1-second periods, S_{M1} (g)	1.31
Design spectral response acceleration at short periods, S_{DS} (g)	1.78
Design spectral response acceleration at 1-second periods, S_{D1} (g)	0.88
Mapped MCE Geometric Mean Peak Ground Acceleration (g)	1.00
Site Coefficient, F_{PGA}	1.00
MCE Geometric Mean Peak Ground Acceleration, PGA_M (g)	1.00
Long period transition-period, T_L	12 sec

MCE_R = Risk-Targeted Maximum Considered Earthquake
MCE = Maximum Considered Earthquake
Latitude: 37.22676; Longitude: -121.97282

4.7 CORROSIVITY CONSIDERATIONS

Two soil samples were collected during the current study and transported under proper chain-of-custody to CERCO Analytical, Inc. for laboratory testing. The samples were tested for redox potential, pH, resistivity, soluble sulfate, and chloride ion concentrations. These tests provide an indication of the corrosion potential of the soil environment on buried concrete structures and metal pipes.

The results are summarized below with the laboratory test results prepared by CERCO Analytical, Inc. contained in Appendix C.

TABLE 4.7-1
Soil Corrosivity Test Results

Sample Number and Depth	Redox Potential (mV)	pH	Resistivity (OHM-CM)	Soluble Sulfate* (mg/kg)	Chloride Ion* (mg/kg)
1-B1 @ 8.5-10 feet	320	7.46	5,300	28	N.D.
1-B3 @ 20-21.5 feet	380	7.47	5,900	32	N.D.

*Results reported on an "as received" basis
N.D – None detected

A corrosion consultant should provide specific design recommendations on corrosion protection for buried metallic lines.

According to the sulfate test results by CERCO, the sulfate ion concentration was reported to range from 28 to 32 mg/kg of water-soluble sulfate (SO_4). The CBC references the American Concrete Institute Manual, ACI 318 (Chapter 4) for concrete requirements. ACI tables provide the following sulfate exposure categories and classes and concrete requirements in contact with soil based upon the exposure risk.

TABLE 4.7-2
Sulfate Exposure Categories and Classes

Sulfate Exposure Category S	Exposure Class	Water- Soluble Sulfate in Soil % by Weight
Not Applicable	S0	$SO_4 < 0.10$
Moderate	S1	$0.10 \leq SO_4 < 0.20$
Severe	S2	$0.20 \leq SO_4 \leq 2.00$
Very Severe	S3	$SO_4 > 2.00$

TABLE 4.7-3
Requirements for Concrete by Exposure Class

Exposure Class	Max w/cm	Min f'c (psi)	Cement Type			Calcium Chloride Admixture
			ASTM C150	ASTM C595	ASTM C1157	
S0	N/A	2500	No Type restriction	No Type restriction	No Type restriction	No restriction
S1	0.5	4000	II[†‡]	IP(MS), IS(<70), (MS)	MS	No restriction
S2	0.45	4500	V[‡]	IP(HS), IS(<70), (HS)	HS	Not permitted
S3	0.45	4500	V + pozzolan or slag[§]	IP(HS) + pozzolan or slag or IS(<70) (HS) + pozzolan or slag[§]	HS + pozzolan or slag[§]	Not permitted

Notes: † For seawater exposure, other types of portland cements with tricalcium aluminate (C_3A) contents up to 10 percent are permitted if the w/cm does not exceed 0.40.

‡ Other available types of cement such as Type III or Type I are permitted in Exposure Classes S1 or S2 if the C_3A contents are less than 8 or 5 percent, respectively.

§ The amount of the specific source of the pozzolan or slag to be used shall not be less than the amount that has been determined by service record to improve sulfate resistance when used in concrete containing Type V cement. Alternatively, the amount of the specific source of the pozzolan or slag to be used shall not be less than the amount tested in accordance with ASTM C1012 and meeting the criteria in ACI 4.5.1

In accordance with the criteria presented above, the test results are classified in the S0 sulfate exposure class. The minimum concrete strength for this exposure class is specified by the CBC in the table above. As minimum requirements, we recommend that Type II cement be used in foundation concrete for structures at the project site and concrete should incorporate a maximum water cement ratio of 0.5 and a minimum compressive strength of 3,000 psi. It should be noted, however, that the structural engineering design requirements for concrete might result in more stringent concrete specifications.

Testing was not completed for all depths of potential embedment. Once more specifics of the proposed improvements are known, we can provide additional testing and/or guidance regarding the exposure risk for sulfates.

4.8 CONCLUSIONS

From a geologic and geotechnical standpoint, the study area appears to be suitable for the proposed development. The main geologic/geotechnical issues to be addressed at the site are listed below. The recommendations in subsequent sections consider these hazards and concerns.

- Presence of expansive soils.
- Presence of shallow groundwater.
- Potential for liquefaction-induced settlement.

5.0 RECOMMENDATIONS

The recommendations included in this report, along with other sound engineering practices, should be incorporated in the design and construction of the project.

5.1 GRADING

Grading operations should meet the requirements of the Supplemental Recommendations (Appendix E) and should be observed and tested by ENGEO's field representative. ENGEO should be notified a minimum of three days prior to grading in order to coordinate its schedule with the grading contractor.

5.1.1 Demolition and Stripping

Site demolition includes the removal of structures, foundations, and buried structures, including abandoned utilities and septic tanks and their leach fields. Debris and soft compressible soils should be also removed from any location to be graded, from areas to receive fill or structures, or those areas to serve as borrow. The depth of removal of such materials should be determined by the Geotechnical Engineer in the field at the time of grading.

The existing vegetation should be removed from areas to receive fill or improvements, or those areas to serve for borrow. Tree roots should be removed down to a depth of at least 3 feet below existing grade. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with grading operations.

All excavations from demolition and stripping below design grades should be cleaned to a firm undisturbed soil surface determined by the Geotechnical Engineer. This surface should then be scarified, moisture conditioned, and backfilled with compacted engineered fill. The requirements for backfill materials and placement operations are the same as for engineered fill. No loose or uncontrolled backfilling of depressions resulting from demolition or stripping is permitted.

5.1.2 Selection of Materials

With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, organically contaminated materials (soil which contains more than 3 percent organic content by weight), and environmentally impacted soils, we anticipate the site soils are suitable for use as engineered fill. Unsuitable materials and debris, including trees with their root balls, should be removed from the project site.

Subject to approval by the Landscape Architect, organically contaminated soil may be stockpiled in approved areas located outside of the grading limits for future placement within landscape areas. Oversized soil or rock materials (those exceeding two-thirds of the lift thickness or 6 inches in dimension, whichever is less) should be removed from the fill and broken down to meet this requirement or otherwise off-hauled.

The Geotechnical Engineer should be informed when import materials are planned for the site. Import materials should be submitted to, and approved by, the Geotechnical Engineer prior to delivery at the site and should conform to the requirements provided in the Supplemental Recommendations.

5.2 EXISTING FILLS

The site is currently occupied by three structures and associated improvements. As such, buried foundation elements and underground utilities may be present on the site.

Existing fills are considered undocumented and should be subexcavated to expose underlying competent native soils that are approved by the Geotechnical Engineer. If in a fill area, the base of the subexcavations should be processed, moisture conditioned, as needed, and compacted in accordance with the recommendations for engineered fill.

5.3 FILL PLACEMENT

Once a suitable firm base is achieved, the exposed non-yielding native surface should be scarified to a depth of 10 inches, moisture conditioned, and recompacted to provide adequate bonding with the initial lift of fill. All fills should be placed in thin lifts, with the lift thickness not to exceed 10 inches or the depth of penetration of the compaction equipment used, whichever is less.

The following compaction control requirements should be applied to onsite expansive (PI>12) materials:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than 2 percentage points above optimum moisture content.
Required Relative Compaction:	Not less than 92 percent.

The following compaction control requirements should be applied to import or low-expansive (PI<12) soils:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than optimum moisture.
Minimum Relative Compaction:	Not less than 95 percent.

Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material. Additional compaction recommendations may be developed during construction based on materials encountered.

5.4 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. In addition, wet soil conditions are anticipated near the bottom of the parking garage excavation. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather.
2. Mixing with drier materials.
3. Mixing with a lime, lime-flyash, or cement product.
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

Options 3 and 4 should be evaluated and approved by ENGEO prior to implementation.

5.5 GRADED SLOPES

In general, graded slopes should be no steeper than 2:1 (horizontal:vertical). All fill slopes should be adequately keyed into firm materials unaffected by shrinkage cracks. If a cut or cut-fill transition occurs within a graded slope, we recommend that it be overexcavated and reconstructed as an engineered fill slope.

5.6 MONITORING AND TESTING

It is important that all site preparations for site grading be done under the observation of the Geotechnical Engineer's field representative. The Geotechnical Engineer's field representative should observe all graded area preparation, including demolition and stripping, following the recommendations contained herein and in the Supplemental Recommendations.

The final grading and foundation plans should be submitted to the Geotechnical Engineer for review.

5.7 FOUNDATION DESIGN

Although the preliminary structural concept and foundation loads have not yet been developed, based on our experience with similar projects, we anticipate the proposed podium structure may be supported on a stiff structural mat foundation.

As previously mentioned, the southern portion of Building 2 will be located outside of the footprint of the subterranean parking garage. The portion of Building 2 that extends outside of the parking garage should be structurally designed to cantilever or span the distance unsupported. If the distance or loading conditions are too great, additional support from drilled piers with interconnected grade beams may be required. We can provide supplemental recommendations if needed.

The proposed podium building will have two levels of subterranean parking that will extend below the design groundwater level. The structure will need to be designed to resist hydrostatic uplift pressures based on the design groundwater level.

5.7.1 Potential Total and Differential Settlement

Assuming the subterranean parking garage extends at least a distance of 20 feet below grade, we recommend that the foundation design consider 1 inch of total and ½ inch of differential settlement associated with liquefaction-induced settlement. The differential settlement may be assumed to occur over a distance of 30 feet or between adjacent column supports, whichever is closer.

5.7.2 Buoyancy Impacts

The garage will be below the 12-foot design groundwater level and will be subject to buoyancy impacts. The foundation should be designed to resist hydrostatic uplift pressures due to the design groundwater level of 12 feet below existing grade. Uplift resistance can be provided by the weight of the foundation elements and the dead loads of the building. The structural engineer should evaluate the buoyancy uplift on the structure and determine if additional resistance is necessary. Viable alternatives for added uplift resistance include hold-down piers or anchors. These can be designed as active or passive systems and we can provide more details as necessary.

5.7.3 Building Pad Treatment

We recommend the subgrade consist of 12 inches of uniform engineered fill. This can be accomplished by subexcavating to pad subgrade followed by scarifying, mixing, moisture conditioning, and compacting the exposed surface to a depth of approximately 12 inches.

If loose/compressible soils are encountered, they should be removed and replaced with compacted engineered fill. Geotextile stabilization fabric may also be recommended in the field.

Considering the shallow groundwater conditions encountered at the site, the exposed subgrade may be near saturation. In addition, the building pad will be susceptible to disturbance under construction equipment loads. The contractor should limit the use of rubber-tired equipment on the subgrade to reduce potential for creation of unstable areas. The contractor should also consider chemical treatment of the subgrade, especially if construction will occur during wet weather. Alternatively, a working pad can be constructed to assist in protecting the subgrade soils.

5.7.4 Structural Mat Foundation

The proposed building can be supported on a conventional mat foundation. The rigid mat should be designed to impose a maximum allowable bearing capacity of 4,000 pounds per square foot (psf) for dead plus long-term live loads. These values may be increased by one-third when considering transient loads, such as wind or seismic. A modulus of subgrade reaction of 75 psi per inch of deflection can be used for engineered fill or native soil. This value represents the modulus of subgrade reaction for a 1 square foot bearing plate.

Resistance to lateral loads may be provided by frictional resistance between the foundation concrete and the subgrade soils and by passive earth pressure acting against the side of the foundation. A coefficient of friction of 0.35 can be used between concrete and the subgrade. Passive pressures can be taken as equivalent to the pressure developed by a fluid having a weight of 300 pounds per cubic foot (pcf).

Localized liquefaction within gravel deposits located below the mat foundation may result in a reduction in bearing capacity and foundation subgrade soil stiffness. To model this condition, we recommend assuming that the localized bearing capacity and stiffness are reduced to zero. This can be modeled by designing the mat foundation to withstand an edge cantilever distance of 6 feet and an interior span distance of 15 feet.

The concrete slabs should be waterproofed, as discussed in a subsequent section. A double-slab drainage system may also be considered to reduce the chance of moisture or water ponding within the lower garage level.

The subgrade material under a mat foundation should be uniform and the mat should be placed neat against the undisturbed soil. The pad subgrade should not be allowed to dry before placing concrete. The pad subgrade should be checked by a representative of ENGEO prior to concrete placement for compliance with these moisture requirements and to confirm the adequacy of the bearing soil.

5.8 BUILDING RETAINING WALLS

We anticipate the underground parking structure will include below-grade retaining walls approximately 20 feet high constructed on a structural mat foundation or continuous footings.

5.8.1 Design Recommendations

The building retaining walls should be designed to resist lateral earth pressures from natural materials and/or backfill and from any surcharge loads. Provided that adequate drainage is included as recommended below, the restrained walls may be designed using an at-rest equivalent fluid pressure of 60 pcf. The design should account for one-half of any vertical surcharge loads applied as a uniform lateral load to the top 10 feet of the wall.

The building walls should have drainage facilities above the design groundwater depth of 12 feet below existing grade to reduce the potential for build-up of hydrostatic pressures. If the walls are not designed with adequate drainage, we recommend adding an additional equivalent fluid pressure of 40 pcf. The wall design should include the additional 40 pcf hydrostatic pressure for depths greater than the design depth to groundwater of 12 feet below ground surface.

We recommend the seismic performance of the basement retaining walls be evaluated using an active equivalent fluid weight of 40 pcf for drained conditions and an active equivalent fluid weight of 80 pcf for undrained conditions, and a seismic increment of 20 pcf, in accordance with Lew, et al. (2010). This evaluation should be separate from the static design using at-rest earth pressures.

Passive pressures acting on foundations may be assumed as 300 pounds per cubic foot (pcf). A coefficient of friction of 0.35 can be used between concrete and the subgrade.

Basement retaining walls should be waterproofed as discussed in a subsequent section.

5.8.2 Wall Drainage

Design details for draining the basement walls above the groundwater level should be determined during the design process. A sump system may be needed for drainage at this elevation unless the storm drain system will allow for gravity connection.

Construct either graded rock drains or geosynthetic drainage composites behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives:

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-1.025) placed directly behind the wall, or

2. A minimum 12-inch-thick layer of washed, crushed rock. Envelop rock in a minimum 6-ounce, nonwoven geotextile filter fabric.

For both types of rock drains:

1. Place the rock drain directly behind the walls of the structure.

2. Extend rock drains from a depth of 12 feet below the ground surface to within 12 inches of the top of the wall.

3. Place a minimum of 4-inch-diameter perforated pipe at the base of the drain material, inside the rock drain and fabric, with perforations placed down.

4. Place pipe at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

ENGEO should review and approve geosynthetic composite drainage systems prior to use.

5.8.3 Backfill

Backfill behind retaining walls should be placed and compacted in accordance with fill placement recommendations. Use light compaction equipment within 5 feet of the wall face. If moderate to heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement. Alternatively, the wall design can incorporate additional surcharge loading to allow moderate to heavy equipment.

5.9 WATERPROOFING

Permanent dewatering is not recommended and the mat foundation or concrete slabs and basement walls should be waterproofed and designed to resist hydrostatic and/or uplift pressures. The waterproofing should be designed by a consultant that specializes in permanent waterproofing construction. Waterstops should be placed at all construction joints.

5.10 SITE RETAINING WALLS

This section is intended for walls, if any, located outside of the main building that are needed for grades separations or landscaping. Unrestrained, drained retaining walls constructed on level ground and up to 6 feet in height may be designed using active equivalent fluid pressures as follows.

TABLE 5.10-1
Active Equivalent Fluid Pressures

Backfill Slope Condition (horizontal:vertical)	Active Pressure (pounds per cubic foot)
Level	40
3:1	50
2:1	60

Site retaining walls should be designed using an allowable bearing capacity of 2,500 psf. Site retaining wall footings should be founded at least 12 inches below adjacent grade. Passive pressures acting on foundations may be assumed as 250 pcf provided the area in front of the retaining wall is level for a distance of at least 10 feet or three times the depth of the foundation, whichever is greater. Unless the surface in front of the wall is confined by a slab or pavement, the upper one foot of soil should be neglected when calculating passive resistance. A coefficient of friction of 0.35 can be used between concrete foundation and the subgrade. Appropriate safety factors against overturning and sliding should be incorporated into the design calculations.

Wall drainage should be included as discussed in a previous section.

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All backfill should be placed in accordance with the recommendations provided above for engineered fill. Light equipment should be used during backfill compaction to reduce possible overstressing of the walls. The foundation details and structural calculations for retaining walls should be submitted for review.

5.11 TEMPORARY EXCAVATIONS

The Contractor should be familiar with applicable local, state, and federal regulations, including the current Occupational Safety and Health Administration (OSHA) Excavation and Trench Safety Standards. It is the responsibility of the Contractor to provide stable, safe trench and construction slope conditions and to follow OSHA safety requirements. Since excavation procedures may be dangerous, it is also the responsibility of the Contractor to provide a trained "competent person" as defined by OSHA to supervise all excavation operations, ensure that all personnel are working in safe conditions and have thorough knowledge of OSHA excavation safety requirements.

Based on the soil data, excavations up to approximately 20 feet deep may generally consider classification of Type C soil in Cal OSHA shoring, sloping, and benching design (i.e., maximum 1½:1 temporary cut slopes). The Geotechnical Engineer should be present during the excavation of site soils to provide geotechnical recommendations as necessary and identify variations in soil conditions as appropriate.

5.12 TEMPORARY SHORING

We anticipate excavations up to 20 feet deep for the parking garage construction. At this time, we anticipate a cantilevered temporary shoring system consisting of drilled or driven soldier piles with lagging will be utilized. If a cantilevered shoring system is not feasible, we can provide supplemental recommendations for a restrained system.

Applicable loading, including surcharges due to traffic, buildings, stockpiles, construction equipment, etc. should be incorporated into shoring design when the surcharge loading is situated above a 1:1 line of projection extending up the bottom of wall. A uniform, horizontal surcharge loading (in units of pounds per square foot) of 50 percent of the vertical surcharge load should be assumed to act over the upper 10 feet of the wall. Appropriate safety factors against overturning and sliding should also be incorporated into the design calculations.

We anticipate that the final temporary shoring design will be based on the contractor's means and methods of construction, including equipment and available shoring materials, as well as other general conditions defined by the project team. Recommendations for a temporary soldier pile and lagging shoring system are provided below.

We recommend the following design parameters be used for cantilevered walls. As noted above, braced or tieback walls will require additional recommendations.

TABLE 5.12-1
Temporary Soldier Pile and Lagging Shoring Design Parameters

Temporary Shoring Design Element	Design Parameter
Active Earth Pressure:	40 pcf (Level backfill conditions) Active earth pressures should be used where existing buildings and critical utilities are situated outside a 1:1 line of projection extending up from the bottom of the wall
At-Rest Earth Pressure:	60 pcf (Level backfill conditions) At-rest earth pressures should be used where existing buildings and critical utilities are situated within a 1:1 line of projection extending up from the bottom of the wall
Passive Earth Pressure:	300 pcf for soil conditions and 500 pcf for bedrock conditions (anticipated below El. 307, approximate), acting on three times the pier diameter provided the soldier pile is backfilled with structural concrete, if drilled. This value may be increased by ⅓ when considering seismic loads.

5.13 TEMPORARY DEWATERING

Based on the anticipated depths of approximately 20 feet for the planned excavation and considering groundwater levels encountered during our field exploration and a design groundwater level of 12 feet, groundwater may be encountered above the bottom of the excavation. Temporary dewatering during construction may be necessary. Assessment of dewatering should be made prior to excavation to determine the level of groundwater control and dewatering necessary to address long-term conditions for the depressed portions of the structure at this site.

Temporary dewatering during construction may be necessary to keep the excavation and working areas reasonably dry. If necessary, dewatering should be performed in a manner such that water levels are maintained not less than 2 feet below the bottom of excavation prior to and continuously during shoring and foundation installation. As the excavations progress, it may be necessary to dewater the soils ahead of the excavation, such as by continuous pumping from sumps, to control the tendency for the bottom of the excavation to heave under hydrostatic pressures and to reduce inflow of water or soil from beneath temporary shoring.

The selection of equipment and methods should be determined by the dewatering designer/contractor. The dewatering system implemented should be selected so as to have minimal impact on the groundwater level surrounding the proposed excavation.

5.14 SECONDARY SLAB-ON-GRADE CONSTRUCTION

This section provides guidelines for secondary slabs such as exterior walkways, steps, and sidewalks. Secondary slabs-on-grade should be constructed structurally independent of the foundation system. This allows slab movement to occur with a reduced potential for foundation distress. Where secondary slab-on-grade construction is anticipated, care must be exercised in attaining a near-saturation condition of the subgrade soil before concrete placement.

Secondary slabs-on-grade should be designed specifically for their intended use and loading requirements. Cracking of conventional slabs should be expected as a result of concrete shrinkage and the expansive soils at the site. Slabs-on-grade should be reinforced for control of cracking, and frequent control joints should be provided to control the cracking. Such reinforcement should be designed by the Structural Engineer. In our experience, welded wire mesh may not be sufficient to control slab cracking. There are numerous measures that can be implemented to improve the performance of exterior slabs. We would be pleased to consult with you in this regard if desired.

Secondary slabs-on-grade not subject to vehicular loads should have a minimum thickness of 4 inches and be underlain by at least 4 inches of clean crushed rock or gravel. Secondary slabs-on-grade that are subject to vehicular loads should have a minimum thickness of 4 inches and be underlain by at least 6 inches of clean crushed rock or gravel. Exterior slabs should be constructed with thickened edges extending at least beneath the crushed rock or gravel into compacted soil to reduce water infiltration. Slabs should slope away from the buildings at a slope of at least 2 percent to prevent water from flowing toward the building.

5.15 PRELIMINARY PAVEMENT DESIGN

Preliminary pavement design is provided based on assumed Traffic Index and subgrade resistance values (R-value). The Traffic Index should be determined by the Civil Engineer or appropriate public agency. The sections provided below should be revised, if applicable, based on R-value tests performed on samples of actual subgrade materials recovered at the time of grading.

Based on the referenced plans prepared by Architectural Technologies, portions of the entry driveway and circular at grade parking area are underlain by the subterranean parking structure while some portions extend outside of the limits of the parking structure. As such, minor settlement of the parking structure may cause minor cracking of pavements in locations that straddle these transition zones. If possible, at-grade improvements should be located such that they are situated entirely within or outside of the limits of the parking garage. We can provide supplemental recommendations at a later date if relocating of surface improvements can't be achieved.

5.15.1 Flexible Pavement

Based on our field exploration, we estimate that site soil will have a resistance (R-value) value of 5. The following preliminary pavement sections have been determined based on an assumed R-value of 5 according to the method contained in the Highway Design Manual by CALTRANS.

TABLE 5.15.1-1
Preliminary Flexible Pavement Design

Traffic Index (TI)	R-Value of 5 (untreated subgrade)	
	AC (inches)	AB (inches)
4.0	2.5	8.0
5.0	3.0	10.0
6.0	3.5	13.0
7.0	4.0	16.0

Notes: AC is asphalt concrete
AB is aggregate base Class 2 Material with minimum R = 78

5.15.2 Rigid Pavements

We developed recommended pavement sections using the Portland Cement Association Thickness Design for Concrete Highway and Street Pavements manual (1995) based on the assumed subgrade soil type. We recommend the following minimum design sections for rigid pavements:

- Use a minimum section of 8 inches of Portland Cement concrete over 8 inches of Class 2 aggregate base.
- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

5.15.3 Pavement Subgrade Preparation

Pavement construction and all materials (hot mix asphalt and aggregate base) should comply with the requirements of the Standard Specifications of the State of California Division of Highways, Town of Los Gatos requirements and the following minimum requirements.

- All pavement subgrades should be scarified to a depth of 10 to 12 inches below finished subgrade elevation, moisture conditioned to at least 2 percentage points above optimum moisture content, and compacted to at least 95 percent relative compaction and in accordance with Town of Los Gatos requirements.
- Subgrade soils should be in a stable, non-pumping condition at the time aggregate baserock materials are placed and compacted. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor and Geotechnical Engineer.

- Aggregate baserock materials should meet current Caltrans specifications for Class 2 aggregate baserock and should be compacted to at least 95 percent of maximum dry density at a moisture content of at least optimum. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented after placement and compaction of the aggregate base. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor and Geotechnical Engineer.

- Adequate provisions must be made such that the subgrade soils and aggregate baserock materials are not allowed to become saturated.

- All concrete curbs separating pavement and irrigated landscaped areas should extend into the subgrade and below the bottom of adjacent aggregate baserock materials. An undercurb drain could also be considered to help collect and transport subsurface seepage.

5.16 DRAINAGE

Perimeter grades should be positively sloped at all times to provide for rapid removal of surface water runoff away from the foundation systems and to prevent ponding of water under foundations or seepage toward the foundation systems at any time during or after construction. Ponded water may cause undesirable soil swell and loss of strength. As a minimum requirement, finished grades should have slopes of at least 5 percent within 10 feet from the exterior walls and at right angles to allow surface water to drain positively away from the structure. For paved areas, the slope gradient can be reduced to 2 percent.

All surface water should be collected and discharged into outlets approved by the Civil Engineer. Landscape mounds must not interfere with this requirement.

All roof stormwater should be collected and directed to downspouts. Stormwater from roof downspouts should not be allowed to discharge directly onto the ground surface in close proximity to the foundation system, such as via spashblocks. Rather, stormwater from roof downspouts should be directed to a solid pipe that discharges into the street or to an outlet approved by the Civil Engineer. If this is not acceptable, we recommend downspouts discharge at least 5 feet away from foundations. Alternatively, engineered stormwater systems can be developed under the guidance of ENGEO.

5.17 STORMWATER INFILTRATION AND BIORETENTION AREAS

Based on the anticipated fines content and laboratory test results, the near-surface site soils are expected to have low permeability values to handle stormwater infiltration. Post-construction BMPs should not rely on infiltration; rather, we recommend BMPs receive subdrains that discharge treated stormwater into the planned storm drain system.

If possible, we recommend bioswales/bioretention areas and other BMPs be planned a minimum of 5 feet away from structural site improvements. Where this is not practical, bioretention areas located within 5 feet of structural onsite or offsite improvements can either:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or

2. Incorporate filter material compacted to between 85 and 90 percent relative compaction (ASTM D1557, latest edition).

In addition, one of the following options should be followed:

1. Bioretention design should incorporate a waterproofing system lining the bioswale excavation and a subdrain, or other storm drain system, to collect and convey water to an approved outlet. The waterproofing system should cover the bioretention area excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.

2. Alternatively, and with increased risk of movement of adjacent improvements, if minor infiltration is desired, the perimeter of the bioretention areas should be lined with an HDPE tree root barrier that extends at least 1 to 2 feet below the bottom of the bioretention area.

In addition, site improvements located adjacent to bioretention areas that are underlain by base rock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement. Where adjacent site improvements include design elements that will experience lateral loads (such as from impact or traffic patterns), additional design considerations may be required.

Given the nature of bioretention systems and possible proximity to improvements, we recommend ENGEO consult further with you as needed, review design plans, and provide testing and observation services during the installation of linings, compaction of the filter material, and connection of designed drains (if implemented).

It should be noted that the contractor is responsible for conducting all excavation and shoring in a manner that does not cause damage to adjacent improvements during construction and future maintenance of the bioretention areas. As with any excavation adjacent to improvements, the contractor should minimize the exposure time such that the improvements are not detrimentally impacted.

5.18 REQUIREMENTS FOR LANDSCAPING IRRIGATION

The geotechnical foundation design parameters contained in this report have considered the swelling potential of some of the site soils; however, it is important to recognize that swell in excess of that anticipated is possible under adverse drainage or irrigation conditions. Therefore, planted areas should be avoided immediately adjacent to the buildings. If planting adjacent to the structure is desired, the use of watertight planter boxes with controlled discharge or the use of plants that require very little moisture is recommended.

Sprinkler systems should not be installed where they may cause ponding or saturation of foundation soils within 5 feet from walls. Such ponding or saturation could result in undesirable soil swell, loss of compaction and consequent foundation and slab movements. Irrigation of landscaped areas should be strictly limited to that necessary to sustain vegetation. The Landscape Architect and prospective owners should be informed of the surface drainage and irrigation requirements included in this report.

5.19 UTILITIES

It is recommended that utility trench backfilling be done under the observation of a Geotechnical Engineer. Ideally, pipe zone backfill (i.e. material beneath and immediately surrounding the pipe) should consist of native material less than ¾ inch in maximum dimension compacted in accordance with recommendations provided above for engineered fill. Trench zone backfill (i.e. material placed between the pipe zone backfill and the ground surface) should also consist of native soil compacted in accordance with recommendations for engineered fill. Controlled density fill is also suitable for pipe zone and trench zone backfill.

If required by local agencies, where import material is used for pipe zone backfill, we recommend it consist of quarry fines, fine- to medium-grained sand, or a well-graded mixture of sand and gravel and that this material not be used within 2 feet of finish subgrades. This material should be compacted to at least 90 percent relative compaction at a moisture content of not less than optimum.

In general, uniformly graded gravel should not be used for pipe or trench zone backfill due to the potential for migration of soil into the relatively large void spaces present in this type of material and for movement of water along trenches backfilled with this type of material. If uniformly graded gravel is used, we recommend that it be encapsulated in 6-ounce filter fabric. Providing outlet locations into manholes or catch basins for water collected in granular trench backfill should also be considered.

All utility trenches entering the buildings and paved areas should be provided with an impervious seal where the trenches pass under or through the building perimeter or curb lines. The impervious plug should extend at least 3 feet to both sides of the crossing and should be placed below, around, and above the utility pipe such that it is entirely in contact with the trench walls and pipe. This is to prevent surface water percolation into the import sand or gravel pipe zone backfill under foundations and pavements where such water would remain trapped in a perched condition.

Care should be exercised where utility trenches are located beside foundation areas. Utility trenches constructed parallel to foundations should be located entirely above a plane extending down from the lower edge of the footing at an angle of 45 degrees. Utility companies and Landscape Architects should be made aware of this information.

Utility trenches in areas to be paved should be constructed in accordance with the Town of Los Gatos requirements or approved alternatives. Compaction of backfill by jetting should not be allowed at this site. If there appears to be a conflict between the Town or other Agency requirements and the recommendations contained in this report, this should be brought to the Owner's attention for resolution prior to submitting bids.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to developers, contractors, buyers, architects, engineers, and designers for the project so that the necessary steps can be taken by the contractors and subcontractors to carry out such recommendations in the field. The conclusions and recommendations contained in this report are solely professional opinions.

The professional staff of ENGEO Incorporated strives to perform its services in a proper and professional manner with reasonable care and competence but is not infallible. There are risks of earth movement and property damages inherent in land development. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of preparation of ENGEO's documents of service. This document must not be subject to unauthorized reuse, that is, reuse without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time. Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims, including, but not limited to claims arising from or resulting from the performance of such services by other persons or entities, and any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

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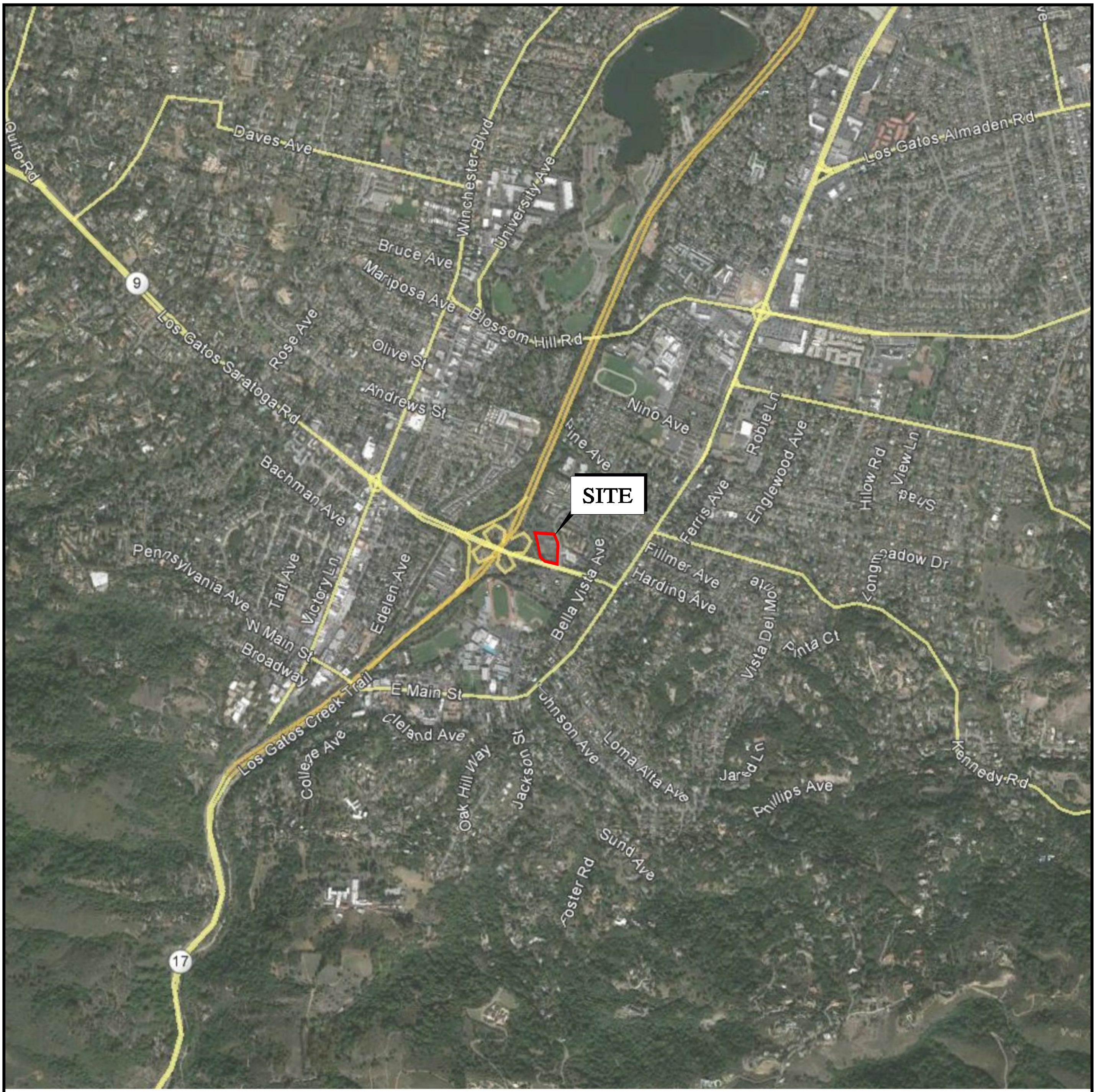
FIGURES



BASE MAP SOURCE: GOOGLE EARTH PRO

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VICINITY MAP
401 ALBERTO WAY
LOS GATOS, CALIFORNIA

PROJECT NO.: 12175.000.000	FIGURE NO. 1
SCALE: AS SHOWN	
DRAWN BY: LL	CHECKED BY: BB

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-B3 BORING

BASE MAP SOURCE: GOOGLE EARTH PRO

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SITE MAP

401 ALBERTO WAY

LOS GATOS, CALIFORNIA

PROJECT NO.: 12175.000.000	
SCALE: AS SHOWN	
DRAWN BY: LL	CHECKED BY: BB

FIGURE NO. 2

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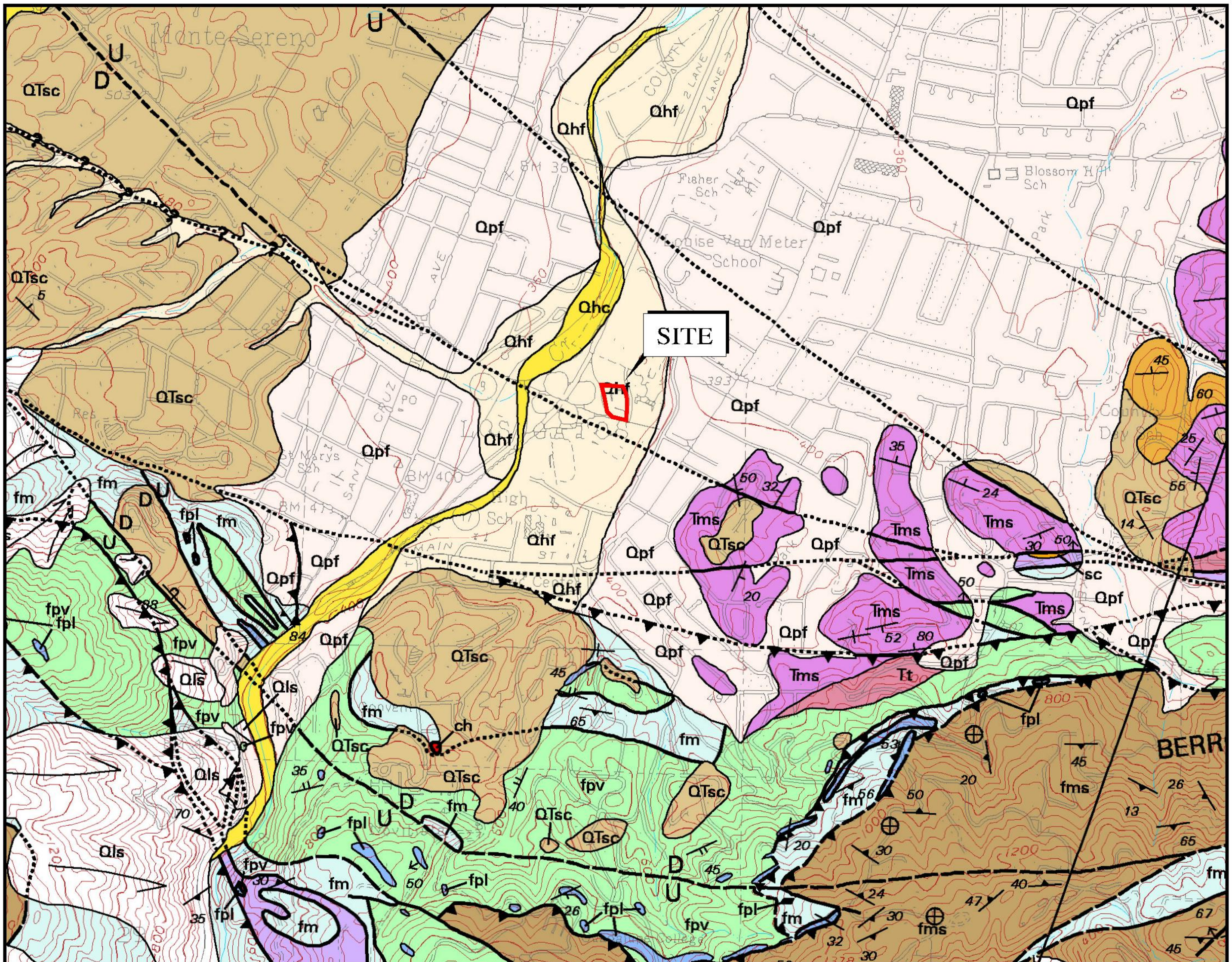
ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

	GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED
	FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL. SAWTEETH ARE ON UPPER PLATE OF LOW ANGLE THRUST FAULT.
	DIRECTION OF LANDSLIDE MOVEMENT

STRIKE AND DIP OF STRATA

INCLINED VERTICAL OVERTURNED

Qls	LANDSLIDE DEPOSITS, UNDIVIDED
Qhc	STREAM CHANNEL DEPOSITS (HOLOCENE)
Qhf	ALLUVIAL FAN DEPOSITS (HOLOCENE)
Qpf	ALLUVIAL FAN DEPOSTS (PLEISTOCENE)
QTsc	SANTA CLARA FORMATION
sc	SILICA-CARBONATE ROCK
Tms	MONTEREY SHALE
Tt	TEMBLOR SANDSTONE
Jos	SERPENTINIZED ULTRAMAFIC ROCKS
fpl	FORAMINFERAL LIMESTONE
fpv	VOLCANIC ROCKS
fms	SANDSTONE
ch	CHERT BLOCKS

N

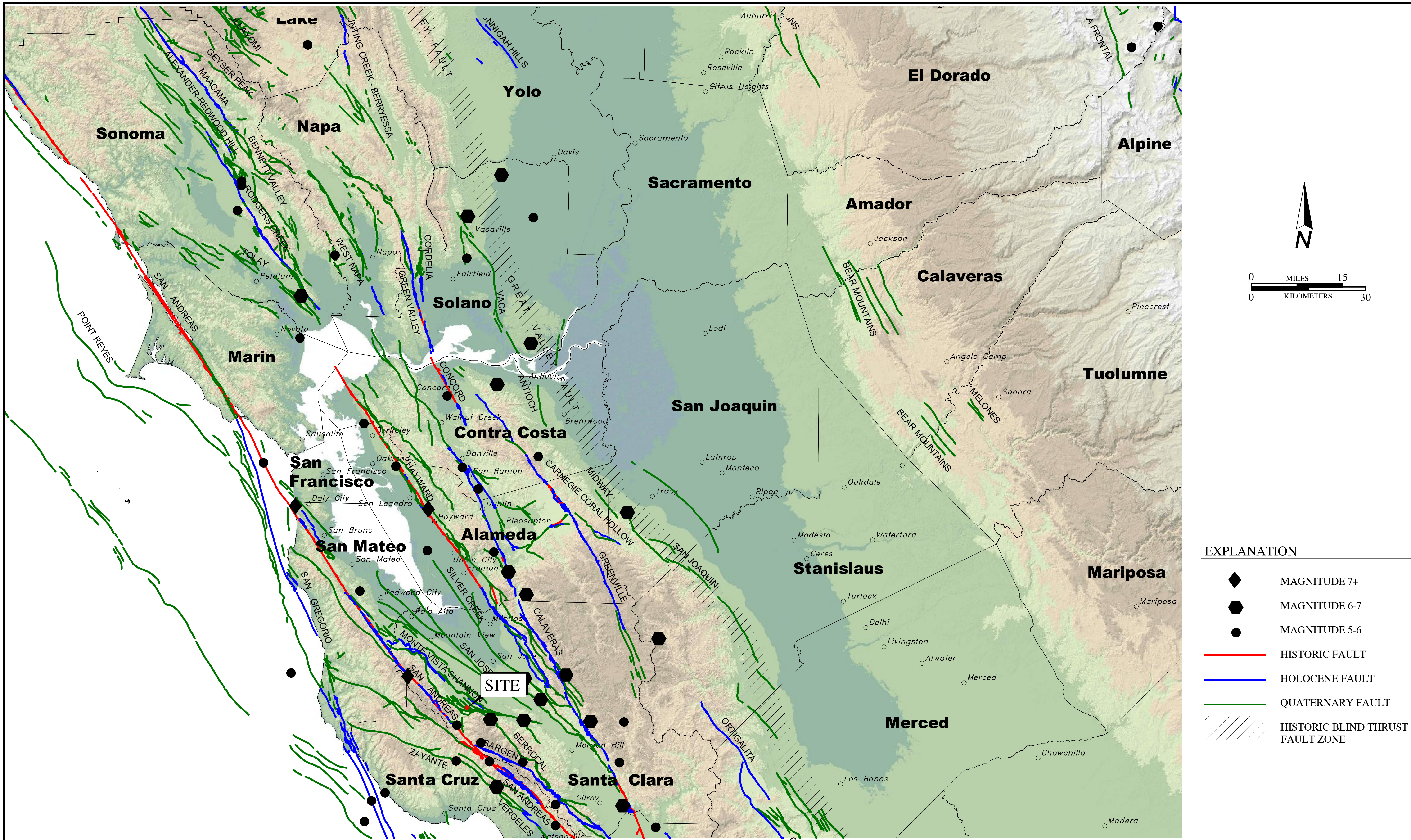
0 FEET 2000
0 METERS 1000

BASE MAP SOURCE: MCLAUGHLIN, 2001

ENGEO Expect Excellence	REGIONAL GEOLOGIC MAP 401 ALBERTO WAY LOS GATOS, CALIFORNIA	PROJECT NO.: 12175.000.000 SCALE: AS SHOWN DRAWN BY: LL CHECKED BY: BB	FIGURE NO. 3

G:\Drafting\DRAFTING2_Dwg_10000 to 12999\12175\000\GEX\12175000000-3-GEX-GEOMAP-0715.dwg Plot Date: 7-17-15 llee

ORIGINAL FIGURE PRINTED IN COLOR



BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

ENGEO
Expect Excellence

REGIONAL FAULTING AND SEISMICITY
401 ALBERTO WAY
LOS GATOS, CALIFORNIA

PROJECT NO.: 12175.000.000	FIGURE NO. 4
SCALE: AS SHOWN	
DRAWN BY: LL	CHECKED BY: BB

ORIGINAL FIGURE PRINTED IN COLOR

G:\Drafting\DRAFTING2_Dwg_10000 to 12999\12175\000\GEX\12175000000-4-GEX-FAULTMAP-0715.dwg Plot Date: 7-10-15 llee

APPENDIX A

Boring Logs
(ENGEO, 2015)

A P P E N D I X A

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures
			GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures
			GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures
			SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures
			SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity
			CL - Inorganic clay with low to medium plasticity
			OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity
			CH - Fat clay with high plasticity
			OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE: 200, 40, 10, 4

CLEAR SQUARE SIEVE OPENINGS: 3/4 ", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

LINE TYPES

________	Solid - Layer Break
– – – – – –	Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

- ▽ Groundwater level during drilling
- ▼ Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

ENGEO Expect Excellence

ENGEO INCORPORATED

LOG OF BORING 1-B1

Geotechnical Exploration
401 Alberto Way
Los Gatos, California
12175.000.000

DATE DRILLED: 6/27/2015
HOLE DEPTH: Approx. 15 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (): Approx. 341½ ft.

LOGGED / REVIEWED BY: I. McCreery / PCG
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3.5 inches AC over 4 inches AB												
5	1		CLAYEY SAND WITH GRAVEL (SC), dark reddish brown, medium dense, slightly moist			17	37	18	19		9.4	98.7	1290.8		UU
10	2, 3		CLAYEY SAND WITH GRAVEL (SC), olive brown, medium dense, slightly moist			19									
15	4		CLAYEY SAND WITH GRAVEL (SC), brown and gray, dense, slightly moist			34									
			Total depth approxiamtely 15 feet bgs. Groundwater not encountered during drilling. Backfilled with grout.												

ENGEO INCORPORATED

LOG OF BORING 1-B2

Geotechnical Exploration
401 Alberto Way
Los Gatos, California
12175.000.000

DATE DRILLED: 6/27/2015
HOLE DEPTH: Approx. 40½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (): Approx. 340 ft.

LOGGED / REVIEWED BY: I. McCreery / PCG
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3 inches AC over 4 inches AB												
	1		CLAYEY SAND WITH GRAVEL (SC), dark reddish brown, medium dense, slightly moist			17					9.9	102.4		1.35	UC
5	2		CLAYEY SAND WITH GRAVEL (SC), dark reddish brown, medium dense, slightly moist			16					7.5	103.4			
			CLAYEY SAND WITH GRAVEL (SC), dark reddish brown, medium dense, slightly moist			15									
10	3					50 for 6 in.									
	4		POORL GRADED SAND WITH GRAVEL (SP), gray and dark reddish brown, very dense, slightly moist												
15	5		CLAYEY SAND WITH GRAVEL (SC), dark reddish brown, dense, moist			44	39	18	21						
20	6		CLAYEY SAND WITH GRAVEL (SC), olive brown, dense, moist		▽	98				12					
25	7		POORLY GRADED GRAVEL WITH CLAY AND SAND (GP-GC), yellowish brown and red, very dense, moist, contains some sandstone rock fragments												

LOG OF BORING 1-B2

ENGEO INCORPORATED

Geotechnical Exploration
401 Alberto Way
Los Gatos, California
12175.000.000

DATE DRILLED:	6/27/2015	LOGGED / REVIEWED BY:	I. McCreery / PCG
HOLE DEPTH:	Approx. 40½ ft.	DRILLING CONTRACTOR:	Britton Exploration
HOLE DIAMETER:	8.0 in.	DRILLING METHOD:	Hollow Stem Auger
SURF ELEV ():	Approx. 340 ft.	HAMMER TYPE:	140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	8		POORLY GRADED GRAVEL (GP-GC), blue gray, very dense, wet			54									
30	9		POORLY GRADED GRAVEL (GP-GC), blue gray, very dense, wet			61									
35	10, 11		SHALE, very dark brown, weak (R2), very closely fractured, damp			101 for 7 in.									
40	12					50 for 6 in.									
			Total depth approximately 40.5 feet bgs. Groundwater encountered at approximately 21 feet during drilling. Backfilled with grout.												

ENGEO INCORPORATED

LOG OF BORING 1-B3

Geotechnical Exploration
401 Alberto Way
Los Gatos, California
12175.000.000

DATE DRILLED: 6/27/2015
HOLE DEPTH: Approx. 34¾ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (): Approx. 338½ ft.

LOGGED / REVIEWED BY: I. McCreery / PCG
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			4.5 inches AC over 1 inch AB												
	1		CLAYEY SAND (SC-CL), dark reddish brown, medium dense, slightly moist			11				47	12.6	102.5		1.03	UC
5	2		CLAYEY SAND WITH GRAVEL (SC), dark reddish brown, dense, slightly moist			44					9	131.2	3260.2		UU
10	3		No Recovery			87									
	4		POORLY GRADED GRAVEL WITH CLAY AND SAND (GP-GC), olive brown, dense, moist			41				11					
15	5		POORLY GRADED GRAVEL WITH CLAY AND SAND (GP-GC), brown and gray, medium dense, moist			18									
20	6		POORLY GRADED GRAVEL WITH CLAY AND SAND (GP-GC), brown, medium dense, wet			28									
	7														
25															

ENGEO INCORPORATED

LOG OF BORING 1-B3

Geotechnical Exploration
401 Alberto Way
Los Gatos, California
12175.000.000

DATE DRILLED: 6/27/2015
HOLE DEPTH: Approx. 34¾ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (): Approx. 338½ ft.

LOGGED / REVIEWED BY: I. McCreery / PCG
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	8		CLAYEY GRAVEL WITH SAND (GC), blue gray, medium dense, wet			27				14					
						157									
30	9		SHALE, very dark brown, weak (R2), very closely fractured												
	10														
			SHALE, very dark brown, weak (R2), very closely fractured			50 for 3 in.									

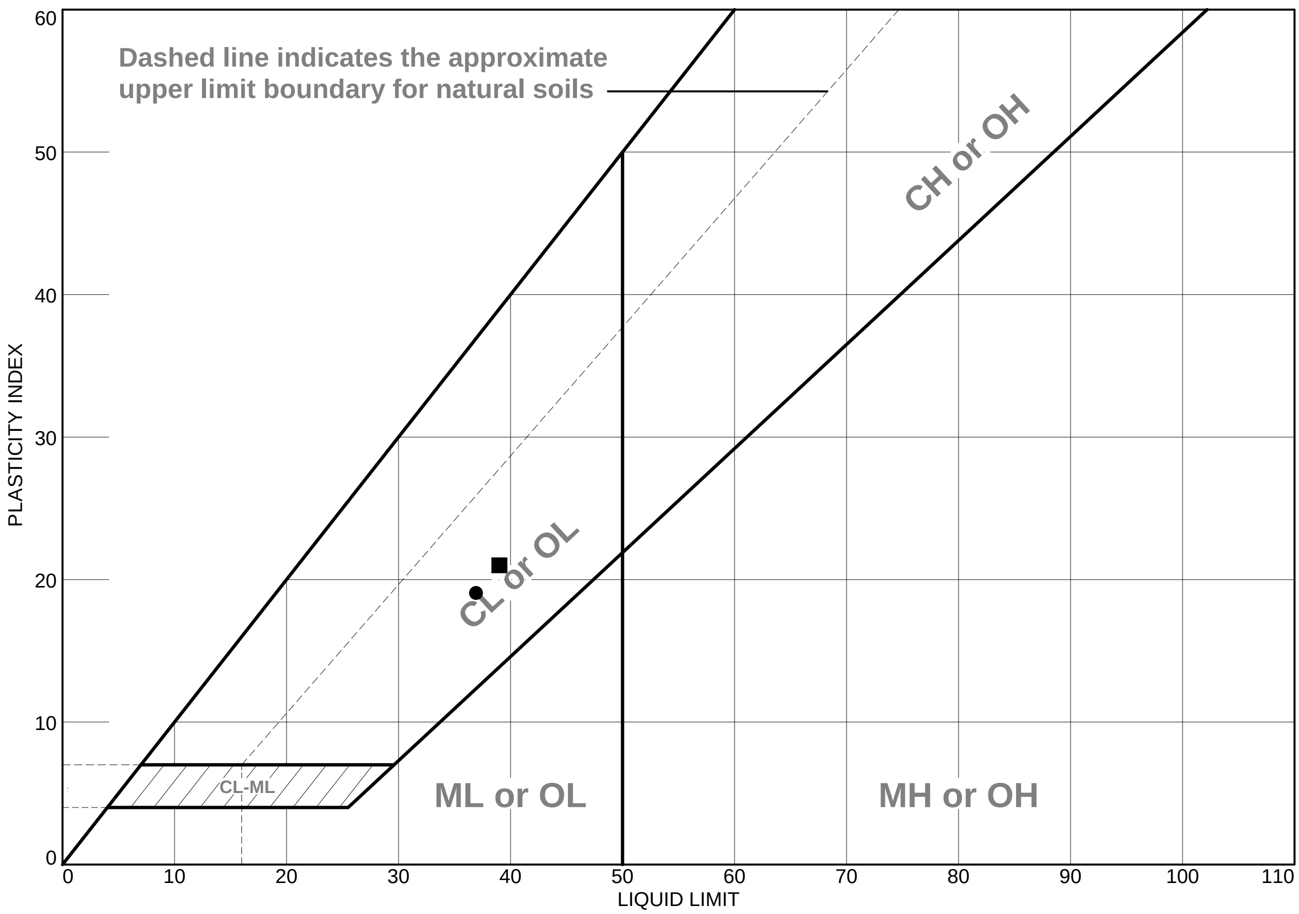
Total depth approximately 34.75 feet bgs. Depth to groundwater not measured due to caving when removing augers. Backfilled with grout.

APPENDIX B

Laboratory Test Results
(ENGEO, 2015)

APPENDIX B

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	37	18	19			
■	See exploration logs	39	18	21			

Project No. 12175.000.000 **Client:** LP Aquisitions, LLC

Project: 401 Alberto Way, Los Gatos, Feasibility Study

●**Depth:** 4.5-5.0 feet **Sample Number:** 1-B1 @ 4.5-5

■**Depth:** 15.0-16.25 feet **Sample Number:** 1-B2 @ 15-16.25

Remarks:

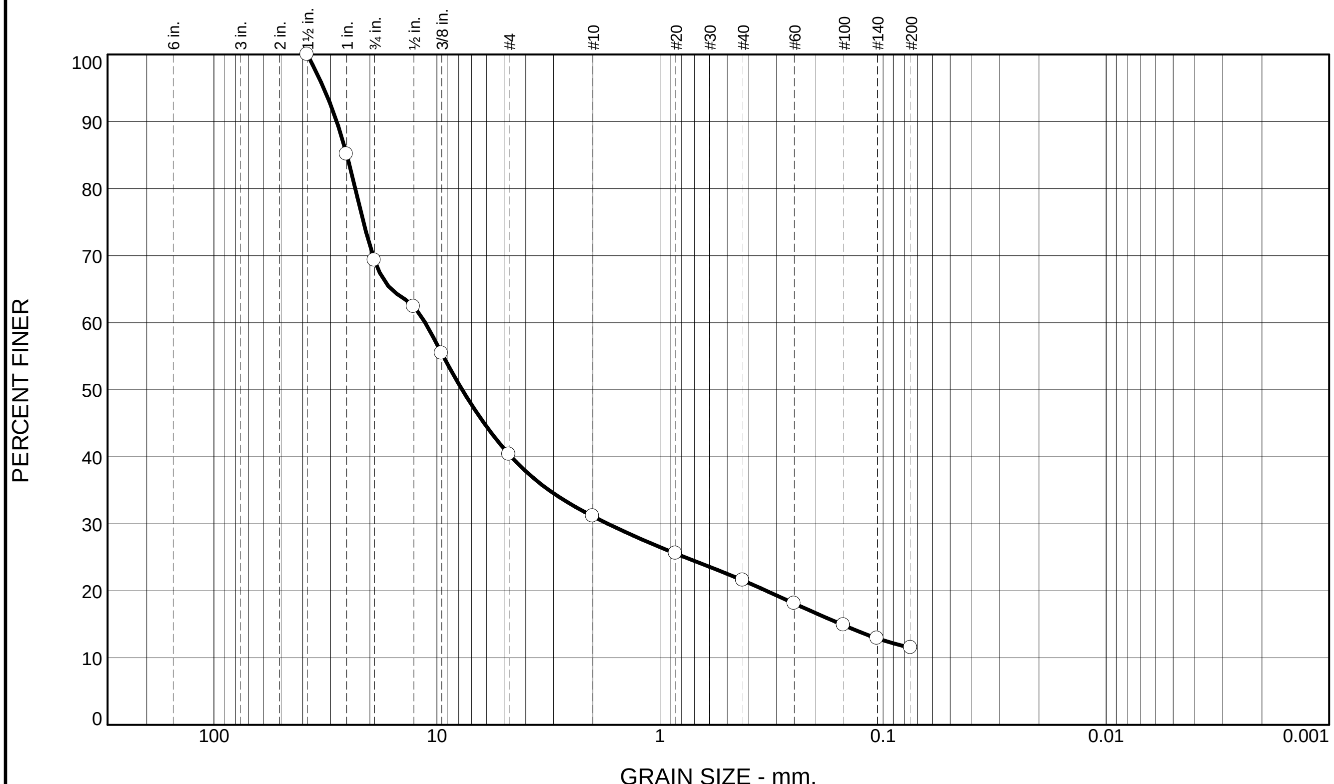
●ASTM D4318, Wet method

■ASTM D4318, Wet method

ENGEO INCORPORATED

Tested By: M. Liu **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	30.7	29.0	9.2	9.6	10.0	11.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2	100.0		
1	85.1		
3/4	69.3		
1/2	62.4		
3/8	55.4		
#4	40.3		
#10	31.1		
#20	25.6		
#40	21.5		
#60	18.1		
#100	14.9		
#140	12.9		
#200	11.5		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 28.1404 D_{85}= 25.3460 D_{60}= 11.2983
D_{50}= 7.7108 D_{30}= 1.7132 D_{15}= 0.1536
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913

Sample Number: 1-B2 @ 21-21.5 **Depth:** 21.0-21.5 feet

Date: 07/07/15

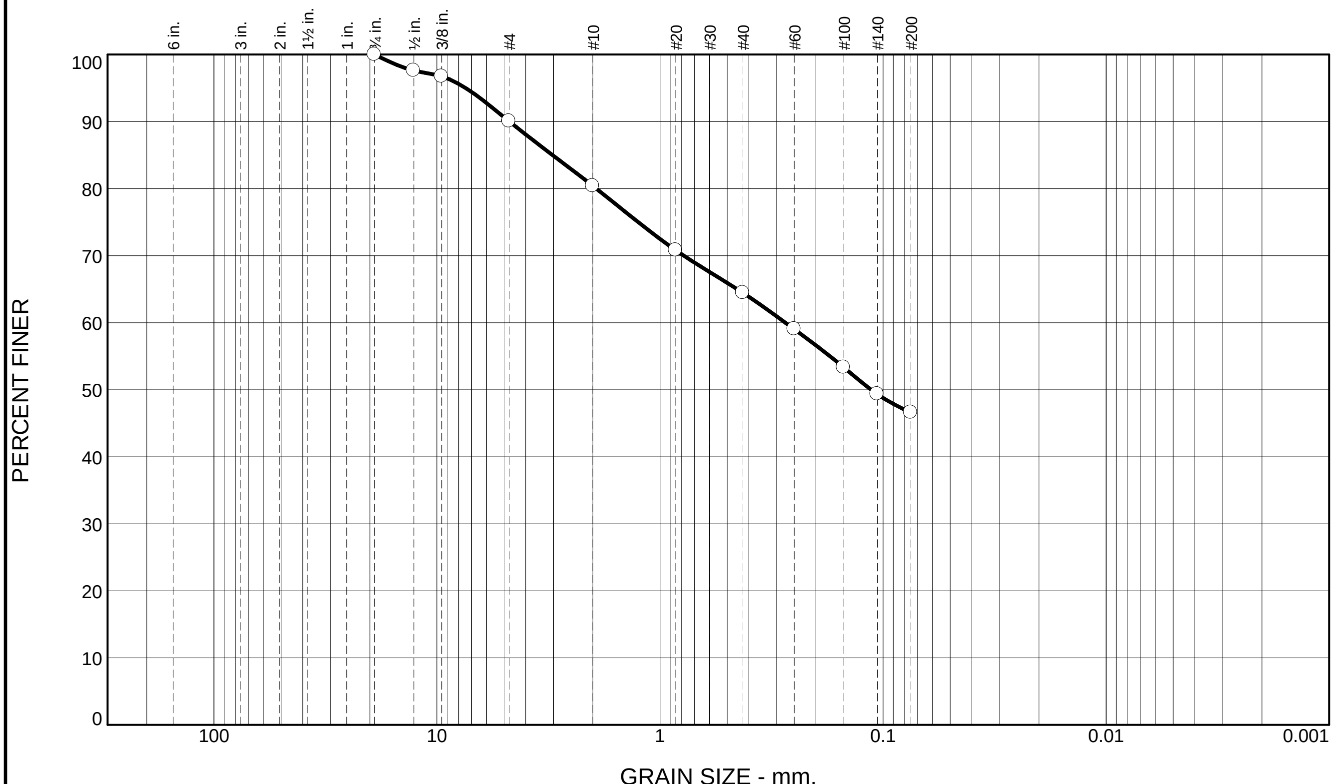
ENGEO INCORPORATED

Client: LP Aquisitions, LLC
Project: 401 Alberto Way, Los Gatos, Feasibility Study

Project No: 12175.000.000

Tested By: J. Lawton **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	10.0	9.6	16.0	17.8	46.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	97.6		
3/8	96.7		
#4	90.0		
#10	80.4		
#20	70.8		
#40	64.4		
#60	59.0		
#100	53.3		
#140	49.3		
#200	46.6		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 4.7364 D_{85}= 3.0312 D_{60}= 0.2740
D_{50}= 0.1132 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913

Sample Number: 1-B3 @ 3-3.5 **Depth:** 3.0-3.5 feet

Date: 07/07/15

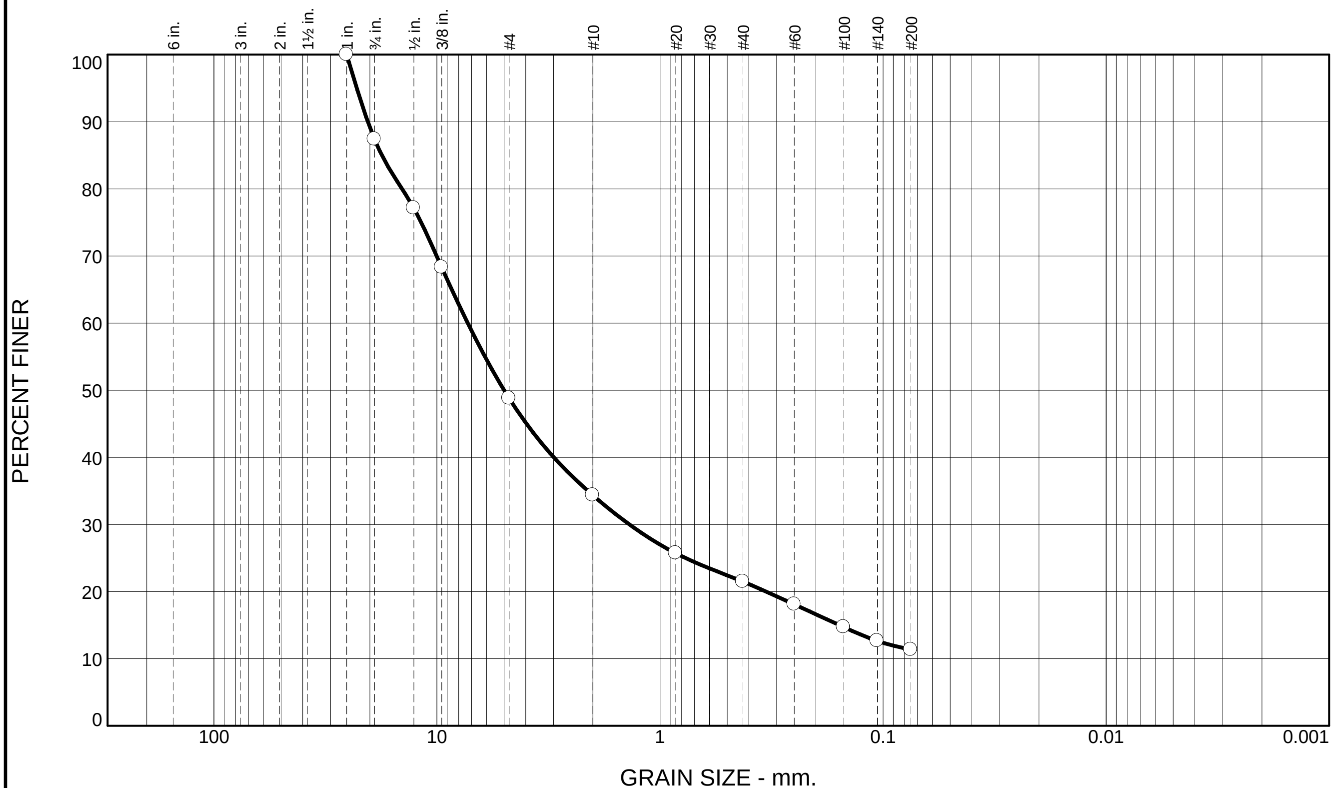
ENGEO INCORPORATED

Client: LP Aquisitions, LLC
Project: 401 Alberto Way, Los Gatos, Feasibility Study

Project No: 12175.000.000

Tested By: J. Lawton **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	12.6	38.6	14.4	12.9	10.2	11.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	87.4		
1/2	77.1		
3/8	68.3		
#4	48.8		
#10	34.4		
#20	25.7		
#40	21.5		
#60	18.1		
#100	14.7		
#140	12.6		
#200	11.3		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 20.4245 D_{85}= 17.6653 D_{60}= 7.2844
D_{50}= 5.0049 D_{30}= 1.3714 D_{15}= 0.1570
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913

Sample Number: 1-B3 @ 11.5-13 **Depth:** 11.5-13.0 feet

Date: 07/07/15

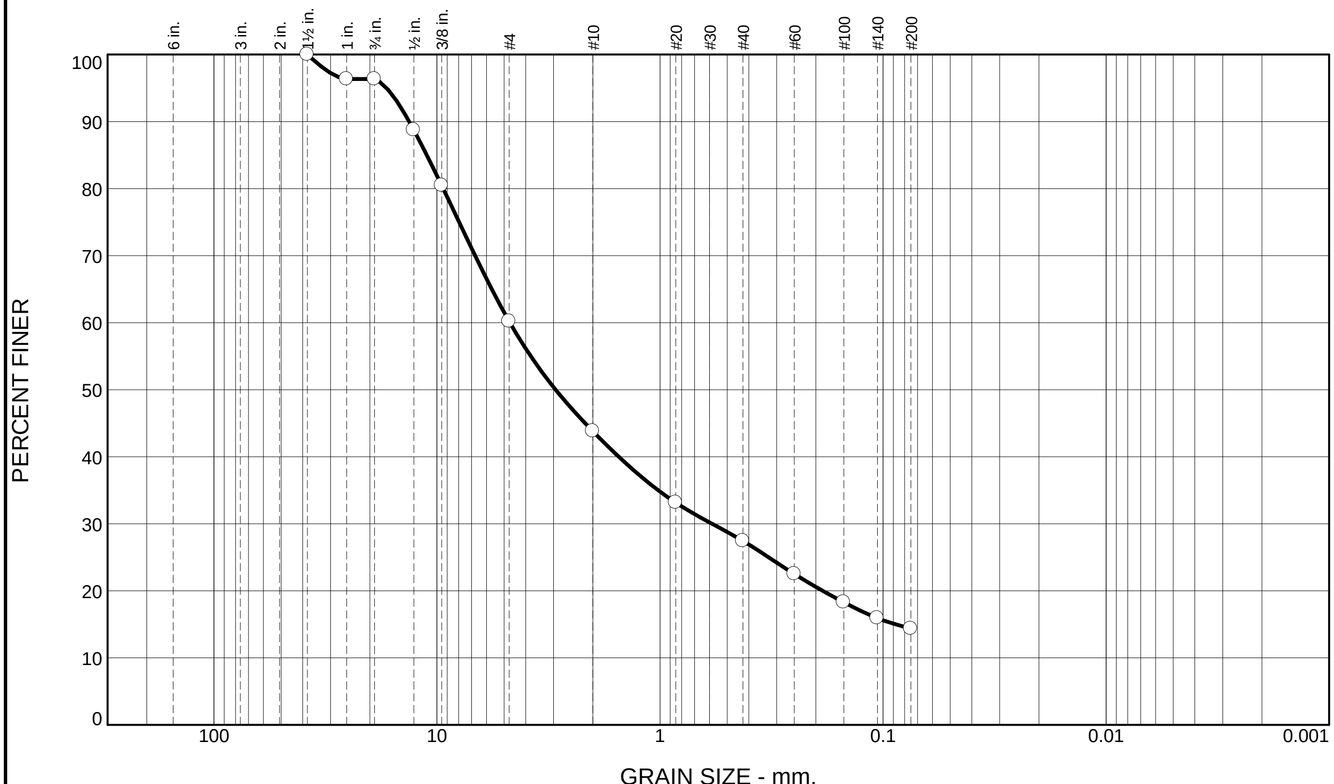
ENGEO INCORPORATED

Client: LP Aquisitions, LLC
Project: 401 Alberto Way, Los Gatos, Feasibility Study

Project No: 12175.000.000

Tested By: J. Lawton **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.7	36.1	16.4	16.4	13.0	14.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2	100.0		
1	96.3		
3/4	96.3		
1/2	88.7		
3/8	80.5		
#4	60.2		
#10	43.8		
#20	33.2		
#40	27.4		
#60	22.5		
#100	18.3		
#140	15.9		
#200	14.4		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 13.3276 D_{85}= 11.1129 D_{60}= 4.7148
D_{50}= 2.9374 D_{30}= 0.5847 D_{15}= 0.0877
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913

Sample Number: 1-B3 @ 25.5-26.5 **Depth:** 25.5-26.5 feet

Date: 07/07/15

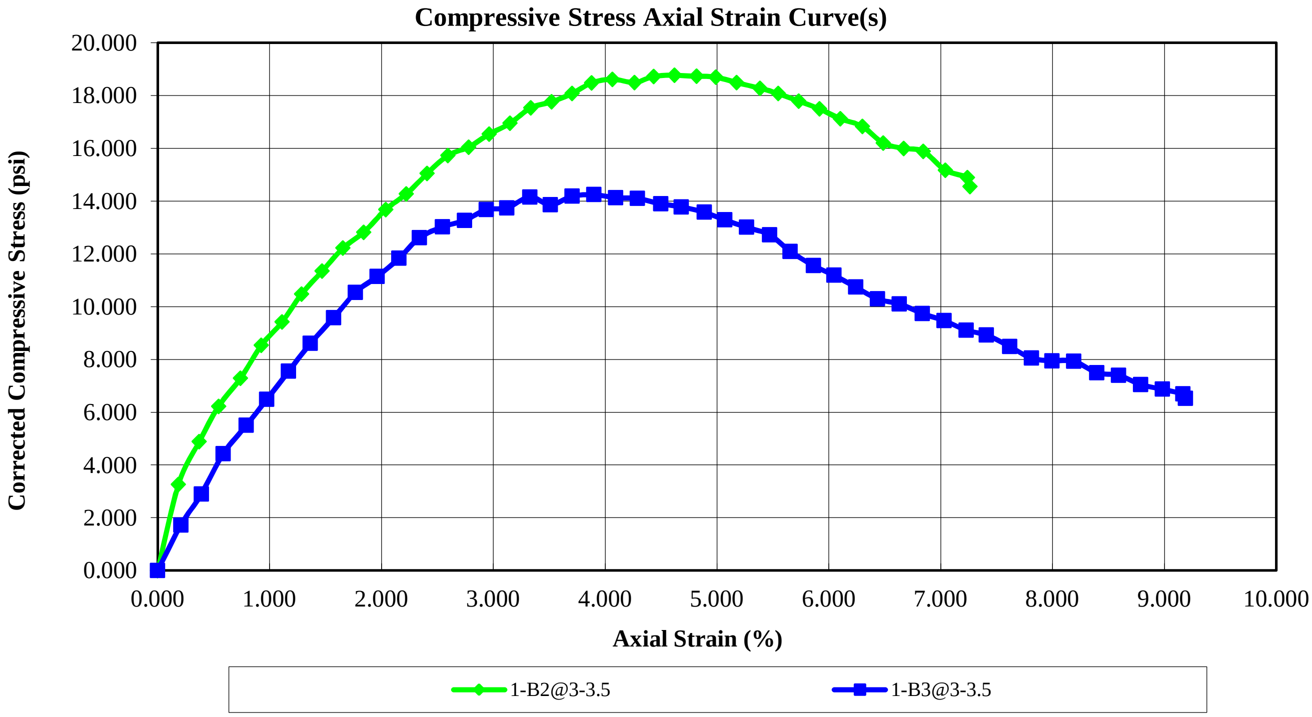
ENGEO INCORPORATED

Client: LP Aquisitions, LLC
Project: 401 Alberto Way, Los Gatos, Feasibility Study

Project No: 12175.000.000

Tested By: J. Lawton **Checked By:** G. Criste

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



	SPECIMEN	
BEFORE TEST	**1-B2@3-3.5**	**1-B3@3-3.5**
Moisture Content (%)	9.9	12.6
Dry Density (pcf)	102.4	102.5
Saturation (%)	42.68	54.25
Void Ratio	0.62	0.61
Diameter (in)	2.397	2.394
Height (in)	5.653	5.372
Height-To-Diameter Ratio	2.358	2.244
TEST DATA		
Unconfined Compressive Strength (psf)	2702.838	2050.721
Undrained Shear Strength (psf)	1351.419	1025.360
Strain Rate (in./min.)	0.05	0.05
Specific Gravity	2.650	2.650
Strain at Failure (%)	4.62	3.90

Test Remarks

SPECIMEN	DESCRIPTION
1-B2@3-3.5	See exploration logs
1-B3@3-3.5	See exploration logs

ENGEO
Expect Excellence

PROJECT NAME: 401 Alberto Way, Los Gatos, Feasibility Study
PROJECT NO: 12175.000.000
CLIENT: LP Aquisitions, LLC
LOCATION: Los Gatos, CA
PHASE NO: 002

Test Date: 07/06/15
Tested By: G. Criste
Reviewed By: D. Seibold

EN GEO

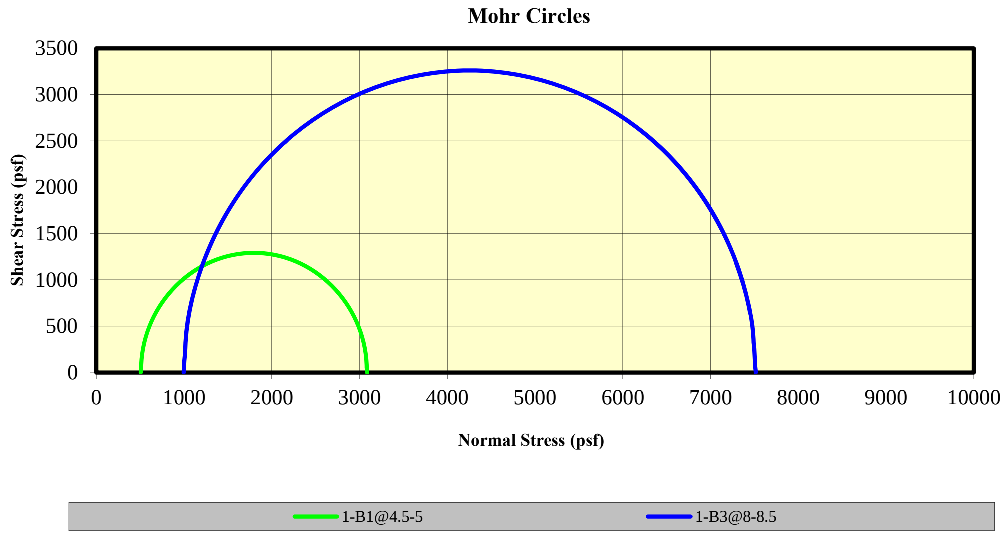
Unconsolidated Undrained Triaxial Test (ASTM D2850)

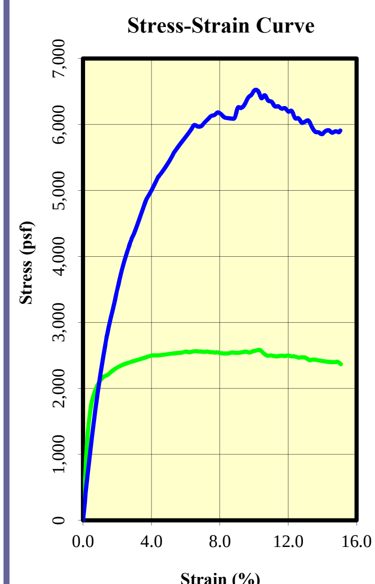
Date: 07/06/15

Checked By: D. Seibold

Date: 07/06/15

Tested By: G. Criste





	Specimen			
Before Test	**1-B1@4.5-5**	**1-B3@8-8.5**		
Water Content (%)	9.38	9.04		
Dry Density (pcf)	98.69	131.24		
Saturation (%)	36.75	80.68		
Void Ratio	0.68	0.31		
Diameter (in)	2.380	2.404		
Height (in)	5.045	5.302		
Liquid Limit	-	-		
Plastic Limit	-	-		
Specific Gravity	2.650	2.750		
Height-to-Diameter Ratio	2.120	2.205		
After Test	**1-B1@4.5-5**	**1-B3@8-8.5**		
Water Content (%)	9.38	9.04		
Saturation (%)	36.75	80.68		
Strain Rate (in/min)	0.05	0.05		
Peak Deviator Stress (psf)	2581.7	6520.3		
Axial Strain @ Failure (%)	10.343	10.038		
	Cell Pressure			
Cell (psf)	504.0	993.6		
Back (psf)	n/a	n/a		
	Principle Stresses at Failure			
σ1 (psf)	3085.7	7513.9		
σ3 (psf)	504.0	993.6		

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			**Cohesion at Failure with a Zero Friction Angle (Ø=0)**			
Cohesion, c (psf)	0.0		1290.8	3260.2		
Friction Angle Ø	0.00		n/a	n/a		

Project Information			
Project Name:	401 Alberto Way, Los Gatos, Feasibility Study		
Project Number:	12175.000.000	Job Number:	12175.000.000
Location:	Los Gatos, CA	Boring Number:	Multiple
Client:	LP Aquisitions, LLC	Sample Number:	Multiple
Description:	See exploration logs		

APPENDIX C

Corrosivity Test Results
(CERCO, 2015)

APPENDIX C

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: ENGEO Incorporated
Client's Project No.: 12175.000.000
Client's Project Name: Alberto Way, Los Gatos
Date Sampled: 27-Jun-15
Date Received: 2-Jul-15
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 13-Jul-2015

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1507016-001	1-B1 @ 8.5'-10'	320	7.46	-	5,300	-	N.D.	28
1507016-002	1-B3 @ 20'-21.5'	380	7.47	-	5,900	-	N.D.	32

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	10-Jul-2015	9-Jul-2015	-	8-Jul-2015	-	10-Jul-2015	10-Jul-2015

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

<u>Quality Control Summary</u> - All laboratory quality control parameters were found to be within established limits

APPENDIX D

Liquefaction Analysis

APPENDIX D

401 Alberto Way, Los Gatos

Liquefaction Evaluation - Youd 2001, Seed 2003, I&B 2008 Methods -

Note, if sloping ground and non-zero statis shear stress exist, user may chose to change value of kalpha

Input

Yellow cells are calculated

Green cells require user input - reference respective papers for details
Corrdction factors on "Driving Force" and "Resisting Force" sheets require user input

Water Table depth at time of Exploration	Water Table depth at time of Liquefaction	amax/g	Mw	$V_{s*40'}$
21	15	1.00	8	1180

* V_{S40} = Avg shear wave velocity in upper 40 feet expressed in ft/s

					At time of Exploration		At time of Liquefaction	
Boring Designation	Depth [ft]	Soil Type	N_m [Blows/ft]	FC	Total Stress [psf]	Effective Stress [psf]	Total Stress [psf]	Effective Stress [psf]
1-B2	15	SC	44	20	1875	1875	1875	1875
1-B2	20	GP-GC	98	12	2500	2500	2500	2188
1-B2	25	GP-GC	54	12	3125	2875.4	3125	2501
1-B2	30	GP-GC	61	12	3750	3188.4	3750	2814
1-B3	15	GP-GC	18	12	1875	1875	1875	1875
1-B3	20	GP-GC	28	12	2500	2500	2500	2188
1-B3	25	GP-GC	27	12	3125	2875.4	3125	2501
						0	0	0
						0	0	0
						0	0	0
						0		0
						0		0
						0		0
						0		0

N_m = Measured SPT Blow Count

YOUD 2001 Methodology Results

Boring Designation	Depth	CRR	CSR	FS
1-B2	15	TDL	0.63	TDL
1-B2	20	TDL	0.71	TDL
1-B2	25	TDL	0.76	TDL
1-B2	30	TDL	0.80	TDL
1-B3	15	0.25	0.63	0.40
1-B3	20	TDL	0.71	TDL
1-B3	25	TDL	0.76	TDL
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!

TDL = Too Dense to Liquefy based on blowcount criteria

ENGEO Expect Excellence 40 years

401 Alberto Way, Los Gatos

SEED 2003 Methodology Results

Boring Designation	Depth	CRR	CSR mean rd	CSR rd + sigma	CSR rd - sigma	Calculated FS mean rd	Calculated FS rd + sigma	Calculated FS rd - sigma
1-B2	15	1.81	0.70	0.75	0.65	FS>2.5	2.43	FS>2.5
1-B2	20	THC	0.84	0.92	0.76	THC	THC	THC
1-B2	25	1.90	0.95	1.05	0.84	2.00	1.80	2.26
1-B2	30	2.88	1.03	1.17	0.90	FS>2.5	2.48	FS>2.5
1-B3	15	0.15	0.69	0.74	0.64	0.22	0.21	0.24
1-B3	20	0.27	0.85	0.93	0.77	0.32	0.30	0.36
1-B3	25	0.23	0.96	1.07	0.86	0.24	0.22	0.27
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

THC = CRR capped at 4, in high seismicity cases, verify

Idriss & Boulanger 2008 Methodology Results

Boring Designation	Depth	CRR	CSR	FS
1-B2	15	THC	-1.53	THC
1-B2	20	THC	0.82	THC
1-B2	25	THC	0.80	THC
1-B2	30	THC	0.84	THC
1-B3	15	0.29	0.71	0.41
1-B3	20	0.93	0.83	1.13
1-B3	25	0.58	0.91	0.64
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!

THC = CRR capped at 4, in high seismicity cases, verify

Liquefaction Evaluation - Driving Force

Boring No.	0

Youd 2001

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	rd	CSR
1-B2	15	1875	1875	0.968	0.63
1-B2	20	2500	2188	0.956	0.71
1-B2	25	3125	2501	0.941	0.76
1-B2	30	3750	2814	0.919	0.80
1-B3	15	1875	1875	0.968	0.63
1-B3	20	2500	2188	0.956	0.71
1-B3	25	3125	2501	0.941	0.76
0	0	0	0	1.000	#DIV/0!
0	0	0	0	1.000	#DIV/0!
0	0	0	0	1.000	#DIV/0!
0	0	0	0	1.000	#DIV/0!
0	0	0	0	1.000	#DIV/0!
0	0	0	0	1.000	#DIV/0!

SEED 2003

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	CN	(N1)60	Ce	Cb	Cr	Cs
1-B2	15	1875	1875	1.03	50	1.1	1.15	0.87	1
1-B2	20	2500	2500	0.89	100	1.1	1.15	0.9	1
1-B2	25	3125	2875	0.83	54	1.1	1.15	0.94	1
1-B2	30	3750	3188	0.79	59	1.1	1.15	0.97	1
1-B3	15	1875	1875	1.03	20	1.1	1.15	0.87	1
1-B3	20	2500	2500	0.89	29	1.1	1.15	0.9	1
1-B3	25	3125	2875	0.83	27	1.1	1.15	0.94	1
0	0	0	0	#DIV/0!	#DIV/0!				
0	0	0	0	#DIV/0!	#DIV/0!				
0	0	0	0	#DIV/0!	#DIV/0!				
0	0	0	0	#DIV/0!	#DIV/0!				
0	0	0	0	#DIV/0!	#DIV/0!				
0	0	0	0	#DIV/0!	#DIV/0!				

DWF
0.91

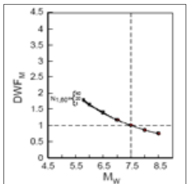
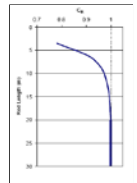


Fig. 13: Recommended Magnitude-Correlated Duration Weighting Factor as a Function of $N_{1,60}$

Rod Length (feet)	CR
9	0.8
12	0.85
15	0.87
18	0.9
21	0.92
24	0.94
27	0.96
30	0.97
33	0.975
36	0.98
39	0.985
45	0.99
60	1
100	1



Boring Designation	Depth	Cfines	(N1)60cs	rd	sigma	rd +sigma	rd -sigma	f	K sigma	K alpha ~
1-B2	15	1.10	55	1.000	0.072	1.072	0.928	0.80	1.02	1.00
1-B2	20	1.05	105	1.000	0.092	1.092	0.908	0.80	0.97	1.00
1-B2	25	1.06	57	1.000	0.111	1.111	0.889	0.80	0.94	1.00
1-B2	30	1.06	63	1.000	0.130	1.130	0.871	0.80	0.92	1.00
1-B3	15	1.08	22	1.000	0.072	1.072	0.928	0.71	1.04	1.00
1-B3	20	1.07	30	1.000	0.092	1.092	0.908	0.75	0.96	1.00
1-B3	25	1.07	29	1.000	0.111	1.111	0.889	0.74	0.92	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00

Boring Designation	mean value of rd				rd + sigma				rd - sigma			
	CSReq	CSRN	CSR*	CSR*α>0	CSReq	CSRN	CSR*	CSR*α>0	CSReq	CSRN	CSR*	CSR*α>0
1-B2	0.65	0.71	0.70	0.70	0.70	0.76	0.75	0.75	0.60	0.66	0.65	0.65
1-B2	0.74	0.81	0.84	0.84	0.81	0.89	0.92	0.92	0.67	0.74	0.76	0.76
1-B2	0.81	0.89	0.95	0.95	0.90	0.99	1.05	1.05	0.72	0.79	0.84	0.84
1-B2	0.87	0.95	1.03	1.03	0.98	1.07	1.17	1.17	0.75	0.83	0.90	0.90
1-B3	0.65	0.71	0.69	0.69	0.70	0.76	0.74	0.74	0.60	0.66	0.64	0.64
1-B3	0.74	0.81	0.85	0.85	0.81	0.89	0.93	0.93	0.67	0.74	0.77	0.77
1-B3	0.81	0.89	0.96	0.96	0.90	0.99	1.07	1.07	0.72	0.79	0.86	0.86
0.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
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~ K alpha = 1.0 for level ground conditions only (no static shear stress)

I&B 2008

MSF
0.88

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	rd	CSR	Csigma	K sigma	K alpha ~	CSR7.5
1-B2	15	1875	1875	0.978	0.64	-12.185	-0.47	1.00	-1.53
1-B2	20	2500	2188	0.966	0.72	-0.119	1.00	1.00	0.82
1-B2	25	3125	2501	0.953	0.77	-1.022	1.10	1.00	0.80
1-B2	30	3750	2814	0.938	0.81	-0.403	1.10	1.00	0.84
1-B3	15	1875	1875	0.978	0.64	0.149	1.02	1.00	0.71
1-B3	20	2500	2188	0.966	0.72	0.224	0.99	1.00	0.83
1-B3	25	3125	2501	0.953	0.77	0.196	0.97	1.00	0.91
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!

~ K alpha = 1.0 for level ground conditions only (no static shear stress)

Liquefaction Evaluation - Resisting Force

Boring No.	0

YOUD 2001

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	CN	Ce	Cb	Cr	Cs	(N1)60
1-B2	15	1875	1875	1.03	1.1333333	1.15	0.95	1	56
1-B2	20	2500	2500	0.89	1.1333333	1.15	0.95	1	109
1-B2	25	3125	2875	0.83	1.1333333	1.15	0.95	1	56
1-B2	30	3750	3188	0.79	1.1333333	1.15	1	1	63
1-B3	15	1875	1875	1.03	1.1333333	1.15	0.95	1	23
1-B3	20	2500	2500	0.89	1.1333333	1.15	0.95	1	31
1-B3	25	3125	2875	0.83	1.1333333	1.15	0.95	1	28
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0	0	0	0	#DIV/0!					#DIV/0!
0	0	0	0	#DIV/0!					#DIV/0!

MSF
0.8474023

Boring Designation	Depth	alpha	beta	(N1)60cs	f	K sigma	K alpha ~	CRR-B	CRR-M
1-B2	15	3.61	1.08	64	0.80	1.02	1.00	0.44	TDL
1-B2	20	1.55	1.03	114	0.80	0.97	1.00	0.82	TDL
1-B2	25	1.55	1.03	59	0.80	0.94	1.00	0.39	TDL
1-B2	30	1.55	1.03	67	0.80	0.92	1.00	0.46	TDL
1-B3	15	1.55	1.03	25	0.73	1.03	1.00	0.30	0.25
1-B3	20	1.55	1.03	34	0.77	0.96	1.00	2.42	TDL
1-B3	25	1.55	1.03	30	0.75	0.93	1.00	0.49	TDL
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~ K alpha = 1.0 for level ground conditions only (no static shear stress)

SEED 2003

Boring Designation	Depth	CRR
1-B2	15	1.81
1-B2	20	74.93
1-B2	25	1.90
1-B2	30	2.88
1-B3	15	0.15
1-B3	20	0.27
1-B3	25	0.23
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0	0	#DIV/0!
0	0	#DIV/0!
0	0	#DIV/0!
0	0	#DIV/0!
0	0	#DIV/0!

I&B 2008

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	(N)60	Ce	Cb	Cr	Cs
1-B2	15	1875	1875	54	1.1333333	1.15	0.95	1
1-B2	20	2500	2500	121	1.1333333	1.15	0.95	1
1-B2	25	3125	2875	67	1.1333333	1.15	0.95	1
1-B2	30	3750	3188	80	1.1333333	1.15	1	1
1-B3	15	1875	1875	22	1.1333333	1.15	0.95	1
1-B3	20	2500	2500	35	1.1333333	1.15	0.95	1
1-B3	25	3125	2875	33	1.1333333	1.15	0.95	1
0	0	0	0	0				
0	0	0	0	0				
0	0	0	0	0				
0	0	0	0	0				
0	0	0	0	0				
0	0	0	0	0				

Iterations of CN Value

Boring Designation	Depth	CN i	N1-1	Cn1	N1-2	Cn2	N1-3	Cn3	N14	CN	(N1)60
1-B2	15	1.01	46	1.02	46	1.02	46	1.02	46	1.02	55
1-B2	20	1.01	46	0.94	46	0.94	46	0.94	46	0.94	114
1-B2	25	0.94	46	0.91	46	0.91	46	0.91	46	0.91	61
1-B2	30	0.95	46	0.88	46	0.88	46	0.88	46	0.88	70
1-B3	15	1.03	22.901497	1.03	22.894161	1.03	22.894248	1.03	22.894247	1.03	23
1-B3	20	0.93	32.194547	0.93	32.0767	0.93	32.070987	0.93	32.07071	0.93	32
1-B3	25	0.88	29.54906	0.88	29.265362	0.87	29.244026	0.87	29.242418	0.87	29
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0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

Boring Designation	Depth	Delta N	(N1)60cs	CRR-B
1-B2	15	4.48	60	#########
1-B2	20	2.07	116	#########
1-B2	25	2.07	63	#########
1-B2	30	2.07	72	#########
1-B3	15	2.07	25	0.29
1-B3	20	2.07	34	0.93
1-B3	25	2.07	31	0.58
0.00	0	0.00	#DIV/0!	#DIV/0!
0.00	0	0.00	#DIV/0!	#DIV/0!
0.00	0	0.00	#DIV/0!	#DIV/0!
0.00	0	0.00	#DIV/0!	#DIV/0!
0.00	0	0.00	#DIV/0!	#DIV/0!
0.00	0	0.00	#DIV/0!	#DIV/0!

APPENDIX E

Supplemental Recommendations

APPENDIX E

SUPPLEMENTAL RECOMMENDATIONS

TABLE OF CONTENTS

GENERAL INFORMATION

PREFACE

These supplemental recommendations are intended as a guide for earthwork and are in addition to any previous earthwork recommendations made by the Geotechnical Engineer. If there is a conflict between these supplemental recommendations and any previous recommendations, it should be immediately brought to the attention of ENGEO. Testing standards identified in this document shall be the most current revision (unless stated otherwise).

DEFINITIONS

Backfill	Soil, rock or soil-rock material used to fill excavations and trenches.
Drawings	Documents approved for construction which describe the work.
The Geotechnical Engineer	The project geotechnical engineering consulting firm, its employees, or its designated representatives.
Engineered Fill	Fill upon which the Geotechnical Engineer has made sufficient observations and tests to confirm that the fill has been placed and compacted in accordance with geotechnical engineering recommendations.
Fill	Soil, rock, or soil-rock materials placed to raise the grades of the site or to backfill excavations.
Imported Material	Soil and/or rock material which is brought to the site from offsite areas.
Onsite Material	Soil and/or rock material which is obtained from the site.
Optimum Moisture	Water content, percentage by dry weight, corresponding to the maximum dry density as determined by ASTM D-1557.
Relative Compaction	The ratio, expressed as a percentage, of the in-place dry density of the fill or backfill material as compacted in the field to the maximum dry density of the same material as determined by ASTM D-1557.
Select Material	Onsite and/or imported material which is approved by the Geotechnical Engineer as a specific-purpose fill.

PART I - EARTHWORK

1.1 GENERAL

1.1.1 WORK COVERED

Supplemental recommendations for performing earthwork and grading. Activities include:

- ✓ Site Preparation and Demolition
- ✓ Excavation
- ✓ Grading
- ✓ Backfill of Excavations and Trenches
- ✓ Engineered Fill Placement, Moisture Conditioning, and Compaction

1.1.2 CODES AND STANDARDS

The contractor should perform their work complying with applicable occupational safety and health standards, rules, regulations, and orders. The Occupational Safety and Health Standards (OSHA) Board is the only agency authorized in the State to adopt and enforce occupational safety and health standards (Labor Code § 142 et seq.). The owner, their representative and contractor are responsible for site safety; ENGEO representatives are not responsible for site safety.

Excavating, trenching, filling, backfilling, shoring and grading work should meet the minimum requirements of the applicable Building Code, and the standards and ordinances of state and local governing authorities.

1.1.3 TESTING AND OBSERVATION

Site preparation, cutting and shaping, excavating, filling, and backfilling should be carried out under the testing and observation of ENGEO. ENGEO shall be retained to perform appropriate field and laboratory tests to check compliance with the recommendations. Any fill or backfill that does not meet the supplemental recommendations shall be removed and/or reworked, until the supplemental recommendations are satisfied.

Tests for compaction shall be made in accordance with test procedures outlined in ASTM D-1557, as applicable, unless other testing methods are deemed appropriate by ENGEO. These and other tests shall be performed in accordance with accepted testing procedures, subject to the engineering discretion of ENGEO.

1.2 MATERIALS

1.2.1 STANDARD

Materials, tools, equipment, facilities, and services as required for performing the required excavating, trenching, filling and backfilling should be furnished by the Contractor.

1.2.2 ENGINEERED FILL AND BACKFILL

Material to be used for engineered fill and backfill should be free from organic matter and other deleterious substances, and of such quality that it will compact thoroughly without excessive voids when watered and rolled.

Unless specified elsewhere by ENGEO, engineered fill and backfill shall be free of significant organics, or any other unsatisfactory material. In addition, engineered fill and backfill shall comply with the grading requirements shown in the following table:

TABLE 1.2.2-1

Engineered Fill and Backfill Requirements

US Standard Sieve	Percentage Passing
3"	100
No. 4	35–100
No. 30	20–100

Earth materials to be used as engineered fill and backfill shall be cleared of debris, rubble and deleterious matter. Rocks and aggregate exceeding the maximum allowable size shall be removed from the site. Rocks of maximum dimension in excess of two-thirds of the lift thickness shall be removed from any fill material to the satisfaction of ENGEO.

ENGEO shall be immediately notified if potential hazardous materials or suspect soils exhibiting staining or odor are encountered. Work activities shall be discontinued within the area of potentially hazardous materials. ENGEO shall be notified at least 72 hours prior to the start of filling and backfilling operations. Materials to be used for filling and backfilling shall be submitted to ENGEO no less than 10 days prior to intended delivery to the site. Unless specified elsewhere by ENGEO, where conditions require the importation of low expansive fill material, the material shall be an inert, low to non-expansive soil, or soil-rock material, free of organic matter and meeting the following requirements:

TABLE 1.2.2-2
Imported Fill Material Requirements

GRADATION (ASTM D-421)	**SIEVE SIZE**	**PERCENT PASSING**
	2-inch	100
	#200	15 - 70
PLASTICITY (ASTM D-4318)	Plasticity Index < 12	
ORGANIC CONTENT (ASTM D-2974)	Less than 2 percent	

A sample of the proposed import material should be submitted to ENGEO no less than 10 days prior to intended delivery to the site.

1.2.3 SUBDRAINS

A subdrain system is an underground network of piping used to remove water from areas that collect or retain surface water or subsurface water. Subsurface water is collected by allowing water into the pipe through perforations. Subdrain systems may drain and discharge to an appropriate outlet such as storm drain, natural swales or drainage, etc.. Details for subdrain systems may vary depending on many items, including but not limited to site conditions, soil types, subdrain spacing, depth of the pipe and pervious medium, as well as pipe diameter.

1.2.3A Pipe

Subdrain pipe shall conform with these supplemental recommendations unless specified elsewhere by ENGEO. Perforated pipe for various depths shall be manufactured in accordance with the following requirements:

TABLE 1.2.3A-1
Perforated Pipe Requirements

Pipe Type	Standard	Typical Sizes (inches)	Pipe Stiffness (psi)
Pipe Stiffness above 200 psi (Below 50 feet of Finished Grade)			
ABS SDR 15.3		4 to 6	450
PVC Schedule 80	ASTM D1785	3 to 10	530
Pipe Stiffness between 100 psi and 150 psi (Between 15 and 50 feet of Finished Grade)			
ABS SDR 23.5	ASTM D2751	4 to 6	150
PVC SDR 23.5	ASTM D3034	4 to 6	153
PVC Schedule 40	ASTM D1785	3 to 10	135
ABS Schedule 40/DWV	ASTM D1527 & D2661	3 to 10	
Pipe Stiffness between 45 psi and 50 psi* (Between 0 to 15 feet of Finished Grade)			
PVC A-2000	ASTM F949	4 to 10	50
PVC SDR 35	ASTM D3034	4 to 8	46
ABS SDR 35	ASTM D2751	4 to 8	45
Corrugated PE	AASHTO M294 Type S	4 to 10	45

*Pipe with a stiffness less than 45 psi should not be used.

Other pipes not listed in the table above shall be submitted for review by the Geotechnical Engineer not less 72 hours before proposed use.

1.2.3B Outlets and Risers

Subdrain outlets and risers must be fabricated from the same material as the subdrain pipe. Outlet and riser pipe and fittings must not be perforated. Covers must be fitted and bolted into the riser pipe or elbow. Covers must seat uniformly and not be subject to rocking.

1.2.3C Permeable Material

Permeable material shall generally conform to Caltrans Standard Specification unless specified otherwise by ENGEO. Class 2 permeable material shall comply with the gradation requirements shown in the following table.

TABLE 1.2.3C-1

Class 2 Permeable Material Grading Requirements

Sieve sizes	Percentage passing
1"	100
3/4"	90 to 100
3/8"	40 to 100
No. 4	25 to 40
No. 8	18 to 33
No. 30	5 to 15
No. 50	0 to 7
No. 200	0 to 3

1.2.3D Filter Fabric

Filter fabric shall meet the following Minimum Average Roll Values unless specified elsewhere by ENGEO.

Grab Strength (ASTM D-4632) .... 180 lbs
Mass per Unit Area (ASTM D-4751)..................................... 6 oz/yd^2
Apparent Opening Size (ASTM D-4751)........... 70-100 U.S. Std. Sieve
Flow Rate (ASTM D-4491)..80 gal/min/ft^2
Puncture Strength (ASTM D-4833) .... 80 lbs

Areas to receive filter fabric must comply with the compaction and elevation tolerance specified for the material involved. Handle and place filter fabric under the manufacturer's instructions. Align and place filter fabric without wrinkles.

Overlap adjacent roll ends of filter fabric in accordance with manufacturer’s recommendations. The preceding roll must overlap the following roll in the direction that the permeable material is being spread. Completely replace torn or punctured sections damaged during placement or repair by placing a piece of filter fabric that is large enough to cover the damaged area and comply with the overlap specified. Cover filter fabric with the thickness of overlying material shown within 72 hours of placing the fabric.

1.2.4 GEOCOMPOSITE DRAINAGE

Geocomposite drainage is a prefabricated material that includes filter fabric and plastic pipe. Filter fabric must be Class A. The drain shall be of composite construction consisting of a supporting structure or drainage core material surrounded by a geotextile. The geotextile shall encapsulate the drainage core and prevent random soil intrusion into the drainage structure. The drainage core material shall consist of a three-dimensional polymeric material with a structure that permits flow along the core laterally. The core structure shall also be constructed to permit flow regardless of the water inlet surface. The drainage core shall provide support to the geotextile.

A geotextile flap shall be provided along drainage core edges. This flap shall be of sufficient width for sealing the geotextile to the adjacent drainage structure edge to prevent soil intrusion into the structure during and after installation. The geotextile shall cover the full length of the core. The geocomposite core shall be furnished with an approved method of constructing and connecting with outlet pipes. If the fabric on the geocomposite drain is torn or punctured, replace the damaged section completely. The specific drainage composite material and supplier shall be preapproved by ENGEO.

The Contractor shall submit a manufacturer's certification that the geocomposite meets the design properties and respective index criteria measured in full accordance with applicable test methods. The manufacturer's certification shall include a submittal package of documented test results that confirm the design values. In case of dispute over validity of design values, the Contractor will supply design property test data from a laboratory approved by ENGEO, to support the certified values submitted.

Geocomposite material suppliers shall provide a qualified and experienced representative onsite to assist the Contractor and ENGEO at the start of construction with directions on the use of drainage composite. If there is more than one application on a project, this criterion will apply to construction of the initial application only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining applications. The soil surface against which the geocomposite is to be placed shall be free of debris and inordinate irregularities that will prevent intimate contact between the soil surface and the drain.

Edge seams shall be formed by utilizing the flap of the geotextile extending from the geocomposite's edge and lapping over the top of the fabric of the adjacent course. The fabric flap shall be securely fastened to the adjacent fabric by means of plastic tape or non-water-soluble construction adhesive, as recommended by the supplier. To prevent soil intrusion, exposed edges of the geocomposite drainage core edge must be covered.

Approved backfill shall be placed immediately over the geocomposite drain. Backfill operations should be performed to not damage the geotextile surface of the drain. Also during operations, avoid excessive settlement of the backfill material. The geocomposite drain, once installed, shall not be exposed for more than 7 days prior to backfilling.

PART II - GEOGRID SOIL REINFORCEMENT

Geogrid soil reinforcement (geogrid) shall be submitted to ENGEO and should be approved before use. The geogrid shall be a regular network of integrally connected polymer tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil or rock. The geogrid structure shall be dimensionally stable and able to retain its geometry under construction stresses and shall have high resistance to damage during construction to ultraviolet degradation and to chemical and biological degradation encountered in the soil being reinforced. The geogrids shall have an Allowable Tensile Strength (T_a) and Pullout Resistance, for the soil type(s) as specified on design plans.

The contractor shall submit a manufacturer's certification that the geogrids supplied meet plans and project specifications. The contractor shall check the geogrid upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the geogrid shall be protected from temperatures greater than 140°F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geogrid will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geogrid damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

Geogrid material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project, for a minimum of three days, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). Geogrid reinforcement may be joined with mechanical connections or overlaps as recommended and approved by the manufacturer. Joints shall not be placed within 6 feet of the slope face, within 4 feet below top of slope, nor horizontally or vertically adjacent to another joint.

The geogrid reinforcement shall be installed in accordance with the manufacturer's recommendations. The geogrid reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed. The geogrid reinforcement shall be placed in continuous longitudinal strips in the direction of main reinforcement. However, if the Contractor is unable to complete a required length with a single continuous length of geogrid, a joint may be made with the manufacturer's approval. Only one joint per length of geogrid shall be allowed. This joint shall be made for the full width of the strip by using a similar material with similar strength. Joints in geogrid reinforcement shall be pulled and held taut during fill placement.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geogrid reinforcement shall be overlapped or mechanically connected where exposed in a wrap around face system, as applicable.

The Contractor may place only that amount of geogrid reinforcement required for immediately pending work to prevent undue damage. After a layer of geogrid reinforcement has been placed, the next succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geogrid reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geogrid reinforcement and soil. Geogrid reinforcement shall be placed to lay flat and pulled tight prior to backfilling. After a layer of geogrid reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geogrid reinforcement in position until the subsequent soil layer can be placed.

Under no circumstances shall a track-type vehicle be allowed on the geogrid reinforcement before at least 6 inches of soil have been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geogrid reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geosynthetic reinforcement at slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided. During construction, the surface of the fill should be kept approximately horizontal. Geogrid reinforcement shall be placed directly on the compacted horizontal fill surface. Geogrid reinforcements are to be placed as shown on plans, and oriented correctly.

PART III - GEOTEXTILE SOIL REINFORCEMENT

The specific geotextile material and supplier shall be preapproved by ENGEO. The contractor shall submit a manufacturer's certification that the geotextiles supplied meet the respective index criteria set when geotextile was approved by ENGEO, measured in full accordance with specified test methods and standards.

The contractor shall check the geotextile upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the geotextile shall be protected from temperatures greater than 140°F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geotextile will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geotextile damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

Geotextile material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project to assist the Contractor and ENGEO personnel at the start of construction. The geotextile reinforcement shall be installed in accordance with the manufacturer's recommendations. The geotextile reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed, secured with staples, pins, or small piles of backfill, placed without wrinkles, and aligned with the primary strength direction perpendicular to slope contours. Cover geotextile reinforcement with backfill within the same work shift. Place at least 6 inches of backfill on the geotextile reinforcement before operating or driving equipment or vehicles over it, except those used under the conditions specified below for spreading backfill.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geotextile reinforcement shall be overlapped or mechanically connected where exposed in a wraparound face system, as applicable.

The contractor may place only that amount of geotextile reinforcement required for immediately pending work to prevent undue damage. After a layer of geotextile reinforcement has been placed, the succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geotextile reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geotextile reinforcement and soil.

Geotextile reinforcement shall be placed to lay flat and be pulled tight prior to backfilling. After a layer of geotextile reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geotextile reinforcement in position until the subsequent soil layer can be placed. Under no circumstances shall a track-type vehicle be allowed on the geotextile reinforcement before at least six inches of soil has been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geotextile reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geotextile reinforcement as slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided.

During construction, the surface of the fill should be kept approximately horizontal. Geotextile reinforcement shall be placed directly on the compacted horizontal fill surface. Geotextile reinforcements are to be placed within three inches of the design elevations and extend the length as shown on the elevation view unless otherwise directed by ENGEO.

Replace or repair any geotextile reinforcement damaged during construction. Grade and compact backfill to ensure the reinforcement remains taut. Geotextile soil reinforcement must be tested to the required design values using the following ASTM test methods.

TABLE III-1

Geotextile Soil Reinforcements

Property	Test
Elongation at break, percent	ASTM D 4632
Grab breaking load, lb, 1-inch grip (min) in each direction	ASTM D 4632
Wide width tensile strength at 5 percent strain, lb/ft (min)	ASTM D 4595
Wide width tensile strength at ultimate strength, lb/ft (min)	ASTM D 4595
Tear strength, lb (min)	ASTM D 4533
Puncture strength, lb (min)	ASTM D 6241
Permittivity, sec^{-1} (min)	ASTM D 4491
Apparent opening size, inches (max)	ASTM D 4751
Ultraviolet resistance, percent (min) retained grab break load, 500 hours	ASTM D 4355

PART IV - EROSION CONTROL MAT

Work shall consist of furnishing and placing a synthetic erosion control mat and/or degradable erosion control blanket for slope face protection and lining of runoff channels. The specific erosion control material and supplier shall be pre-approved by ENGEO.

The Contractor shall submit a manufacturer's certification that the erosion mat/blanket supplied meets the criteria specified when the material was approved by ENGEO. The manufacturer's certification shall include a submittal package of documented test results that confirm the property values. Jute mesh shall consist of processed natural jute yarns woven into a matrix, and netting shall consist of coconut fiber woven into a matrix. Erosion control blankets shall be made of processed natural fibers that are mechanically, structurally, or chemically bound together to form a continuous matrix that is surrounded by two natural nets.

The Contractor shall check the erosion control material upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the erosion mat shall be protected from temperatures greater than 140°F, mud, dirt, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the erosion mat/blanket shall be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be removed by cutting out a section of the mat. The remaining ends should be overlapped and secured with ground anchors. Any erosion mat/blanket damaged during storage or installation shall be replaced by the Contractor at no additional cost to the Owner.

Erosion control material suppliers shall provide a qualified and experienced representative onsite, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). The erosion control material shall be placed and anchored on a smooth graded, firm surface approved by the Engineer. Anchoring terminal ends of the erosion control material shall be accomplished through use of key trenches. The material in the trenches shall be anchored to the soil on maximum 1½ foot centers. Topsoil, if required by construction drawings, placed over final grade prior to installation of the erosion control material shall be limited to a depth not exceeding 3 inches.

Erosion control material shall be anchored, overlapped, and otherwise constructed to ensure performance until vegetation is well established. Anchors shall be as designated on the construction drawings, with a minimum of 12 inches length, and shall be spaced as designated on the construction drawings, with a maximum spacing of 4 feet.

GEOTECHNICAL EXPLORATION

401 ALBERTO WAY
LOS GATOS, CALIFORNIA

Submitted to:
Mr. Shane Arters
LP Acquisitions, LLC
% Lamb Partners, LLC
525 Middlefield Road, Suite 118
Menlo Park, CA 94025

Prepared by:
ENGEO Incorporated

July 17, 2015
Revised August 13, 2015

Project No:
12175.000.000

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Mr. Shane Arters
LP Acquisitions, LLC
℅ Lamb Partners, LLC
525 Middlefield Road, Suite 118
Menlo Park, CA 94025

Subject: 401 Alberto Way
Los Gatos, California

GEOTECHNICAL EXPLORATION

Dear Mr. Arters:

As requested, we completed this geotechnical exploration for your proposed office building project in Los Gatos, California. The accompanying report presents our field exploration and laboratory testing with our conclusions and recommendations regarding the proposed development at the site.

Our findings indicate that the site is suitable for the proposed development provided the recommendations and guidelines provided in this report are implemented during project planning, design and construction. We are pleased to have been of service to you on this project and are prepared to consult further with you and your design team as the project progresses.

Sincerely,

ENGEO Incorporated

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this geotechnical report, as described in our proposal dated June 16, 2015, is to provide design-level geotechnical recommendations associated with the proposed office building development of the site.

We performed the following services:

- Review of available literature, previous reports and geologic maps for the study area.
- Subsurface exploration consisting of three soil borings.
- Laboratory testing of materials sampled during the field exploration.
- Geotechnical data analyses.
- Report preparation summarizing our conclusions and recommendations for the proposed development.

Our services are based on the following plan set:

- A Planning Application for 401-409 Alberto Way, Los Gatos, prepared by Architectural Technologies and dated May 15, 2015.

We prepared this report exclusively for LP Acquisitions, LLC and their design team consultants. ENGEO should review any changes made in the character, design or layout of the development to modify the conclusions and recommendations contained in this report, as necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without the express written consent of ENGEO.

1.2 SITE LOCATION AND DESCRIPTION

The roughly 2.15-acre property is located at 401 Alberto Way in Los Gatos, California. The site is generally bounded by residential development to the north, Los Gatos Saratoga Road to the south, Highway 17 to the west, and Alberto Way to the east (Figures 1 and 2). Based on a recent site visit, the project area is currently occupied by three 2-story office buildings with associated at-grade parking and landscape areas.

1.3 PROPOSED DEVELOPMENT

Based on the referenced plan prepared by Architectural Technologies (dated May 15, 2015), we anticipate the new development will consist of a podium structure including two 2-story office buildings encompassing areas of 47,800 square feet (Building 1) and 45,000 square feet (Building 2) over a two-level below-grade parking garage. The parking garage is shown to underlie the entirety of Building 1 and the majority of Building 2, with the exception of the southern portion of Building 2. Associated improvements include an at-grade parking area, trash enclosure, and landscaped areas. Based on conversations with you, it is our understanding that the office buildings will consist of steel-framed construction.

1.4 AERIAL PHOTOGRAPH REVIEW

We reviewed individual aerial photographs of the site dated 1939, 1948, 1950, 1956, 1968, 1974, 1982, 1993, 1998, 2005, 2006, 2009, 2010 and 2012 provided by Environmental Data Resources (EDR).

The site appears to be vacant land with some vegetation and agricultural use until the time of the photograph dated 1968, at which time three structures with associated paved parking areas are first visible within the site. The site resembles present-day conditions throughout the remaining photographs reviewed.

Additionally, we reviewed numerous stereo-paired images (dated 1937 through 2005) to investigate potential geologic hazards impacting the subject site. Observations made from examining the stereo-paired images were utilized in our geologic review and are discussed in the appropriate sections below.

1.5 PREVIOUS GEOTECHNICAL STUDIES ON NEIGHBORING PROPERTIES

1.5.1 55 Los Gatos Saratoga Road, Earth Systems Geotechnologies (ESG)

55 Los Gatos Saratoga Road, which is located immediately east of the subject site on the opposite side of Alberto Way, was explored by ESG in 2008 for a proposed office building and parking lot. ESG's subsurface exploration consisted of advancing two borings to depths of approximately 41 and 19½ feet below the ground surface (bgs). The borings generally encountered very dense sands and gravels with varying clay content to a depth of approximately 33½ feet bgs, below which depth shale bedrock was observed. Groundwater was encountered by ESG at depths ranging between approximately 18½ and 21 feet bgs. These subsurface findings were utilized in our review of geologic hazards, as discussed in the sections below.

2.0 GEOLOGIC CONDITIONS

2.1 REGIONAL GEOLOGY

Regional geologic mapping by McLaughlin et al. (2000, Figure 3) identifies Holocene-age alluvial fan deposits (Qhf) underlying the site. Similarly, regional mapping by Dibblee (2005) indicates the site is underlain by Quaternary-age sand and gravel of major stream channels (Qg), presumably deposited by nearby Los Gatos Creek.

2.2 REGIONAL FAULTING AND SEISMICITY

Regional geologic mapping by McLaughlin et al. (2001) depicts a concealed splay of the Berrocal fault approximately 200 feet to the south of the site, trending in a direction roughly parallel to Los Gatos Saratoga Road. Similarly, the Fault Lineament & Coseismic Deformation Map for the Town of Los Gatos General Plan Update (Nolan Associates, 1999) depicts the same concealed splay approximately 250 to 300 feet south of the site.

The site is not located within a State of California Earthquake Fault Hazard Zone (Los Gatos Quadrangle, 1991) for active faults, and no known faults cross the site. However, the southern two-thirds of site is located within a Santa Clara County Fault Rupture Hazard Zone (2012) due to the nearby mapped trace of the Berrocal fault to the south of the site, which is identified as a Quaternary-age fault by the USGS (USGS, Quaternary Fault and Fold Database). Additionally, the site is located within a zone for high fault rupture hazard potential as depicted on the Fault Rupture Hazard Zones Map for the Town of Los Gatos General Plan Update (Nolan Associates, 1999). Review of the Fault Lineament & Coseismic Deformation Map for the Town of Los Gatos General Plan Update (Nolan Associates, 1999) and Plate 1 of the USGS Open File Report 95-820 (Schmidt et al., 1995) indicates that the site is not located in an area that experienced a concentration of coseismic ground deformation or damage to the ground surface as a result of the 1989 Loma Prieta Earthquake.

Nearby active[1] and potentially active faults include the Berrocal fault, located approximately 200 to 300 feet south and 0.3 mile north of the site; Monte Vista-Shannon fault located approximately 1.4 miles north of the site; and the San Andreas fault, located approximately 3.4 miles southwest of the site.

Because of the presence of nearby active faults, the Bay Area Region is considered seismically active. Numerous small earthquakes occur every year in the region, and large (>M7) earthquakes have been recorded and can be expected to occur in the future. Figure 4 shows the approximate locations of these faults and significant historic earthquakes recorded within the Greater Bay Area Region.

3.0 FIELD EXPLORATION

3.1 EXPLORATORY BORINGS

The field exploration for this study included advancing three exploratory borings within the project site on June 27, 2015. The borings were drilled to depths ranging from approximately 15 feet bgs to 40½ feet bgs using a track-mounted rig equipped with either 8-inch-diameter hollow-stem augers or 6-inch-diameter solid flight augers. Figure 2 presents the approximate locations of the exploratory borings obtained by taping or pacing from existing features. As a

[1] An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,000 years) (Hart, 1997).

result, the mapped locations should be considered only as accurate as the methods used to determine them.

The borings were logged in the field and soil samples were collected using either a 2½-inch inside diameter (I.D.) California-type split-spoon sampler fitted with 6-inch-long brass liners or a 2-inch outside diameter (O.D.) Standard Penetration Test split-spoon sampler. The penetration of the samplers into the native materials was recorded as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring logs record blow count results as the actual number of blows required for the last 1 foot of penetration; no conversion factors have been applied. The samplers were driven with a 140-pound hammer falling a distance of 30 inches employing an automatic hammer system. The field logs were then used to develop the report boring logs, which are presented in Appendix A.

The boring logs depict subsurface conditions within the borings at the time of the exploration. Subsurface conditions at other locations may differ from conditions occurring at these boring locations, and the passage of time may result in altered subsurface conditions. In addition, stratification lines represent the approximate boundaries between soil types, and the transitions may be gradual.

Upon completion, the test holes were backfilled with grout.

3.2 LABORATORY TESTING

We performed the following laboratory tests on select samples recovered during boring operations:

TABLE 3.2-1
Laboratory Testing

Soil Test	Testing Method	Location of Results
Natural Unit Weight and Moisture Content	ASTM D7263	Appendix A
Atterberg Limits	ASTM D4318	Appendix B
Grain Size Distribution	ASTM D422	Appendix B
Unconfined Compression	ASTM D2166	Appendix B
Unconsolidated Undrained Triaxial	ASTM D2850	Appendix B
Corrosivity Testing (Redox, pH, Resistivity, Chloride, Sulfate)	ASTM D-1498, D-4972, G57, D-4658M, D-4327	Appendix C

The laboratory test results are shown on the borelogs (Appendix A), with individual test results presented in Appendices B and C.

3.3 SUBSURFACE CONDITIONS

In general, our exploratory borings encountered medium dense to dense clayey sands to depths ranging between 10 to 21 feet bgs, which in turn were underlain by medium dense to very dense clayey gravels to depths of approximately 29 to 33 feet bgs. Bedrock consisting of a weak, closely fractured shale was encountered below the gravelly soils. Similar soils and depth to bedrock was observed by ESG on the neighboring property to the east at 55 Los Gatos Saratoga Road.

3.4 GROUNDWATER

Groundwater was encountered during our subsurface exploration and during the exploration by ESG on the neighboring property to the east at depths of approximately 18½ to 21 feet bgs. Plate 1.2 of the Seismic Hazard Zone Report for the Los Gatos Quadrangle (2002) indicates historic groundwater highs between approximately 10 to 20 feet below the ground surface.

Fluctuations in groundwater levels should be expected during seasonal changes or over a period of years because of precipitation changes, perched zones, changes in drainage patterns, and irrigation.

4.0 GEOLOGIC AND GEOTECHNICAL HAZARDS

The site was evaluated with respect to known geologic and other hazards common to the area. The primary hazards and the risks associated with these hazards with respect to the planned development are discussed in the following sections of this report.

4.1 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, soil liquefaction, and lateral spreading. These hazards are discussed in the following sections. Based on topographic data, risk from tsunamis or seiches is considered low to negligible at the site.

4.1.1 Ground Rupture

As described above, the site is not located within a State of California Earthquake Fault Hazard Zone (Los Gatos Quadrangle, 1991) and no known faults cross the site. Review of aerial images provided by EDR and stereo-paired images did not reveal any visible lineaments in the vicinity or crossing the subject site. Review of the Fault Lineament & Coseismic Deformation Map for the Town of Los Gatos General Plan Update (Nolan Associates, 1999) and Plate 1 of the USGS Open File Report 95-820 (Schmidt et al., 1995) indicates that the site is not located in an area that experienced a concentration of coseismic ground deformation or damage to the ground surface as a result of the 1989 Loma Prieta Earthquake.

However, the site is located within a Santa Clara County Fault Rupture Hazard Zone and high fault rupture hazard potential zone (Nolan Associates, 1999) due to the nearby mapped trace of the Berrocal fault, located approximately 200 to 300 feet south of the site. The Berrocal fault, which trends in an east-west direction in the project area, is a southwest-dipping, reverse dextral-oblique fault zone (USGS Quaternary Fault and Fold Database). Should the fault zone pass through the subject site, a significant vertical offset of the geologic contact between bedrock and overlying sediments would be expected across the northern and southern portions of the site. However, Borings 1-B2 and 1-B3, advanced roughly 300 feet apart on the southern and northern sides of the site, respectively, encountered shale bedrock at approximately the same elevation (between approximately El. 307 and 309.5). Bedrock was encountered at a similar depth by ESG in a boring advanced at a neighboring property immediately east of the site.

Based on the absence of observable photo lineaments in the vicinity or crossing the site, consistently mapped location of the Berrocal fault to the south of the site, and lack of coseismic deformation observed at the site and in the immediate vicinity of the site following the 1989 Loma Prieta Earthquake, it is our opinion that ground rupture is unlikely at the subject property.

4.1.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the latest California Building Code (CBC) requirements, as a minimum.

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

4.1.3 Soil Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded fine sands below the groundwater table. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop and liquefaction of susceptible soil to occur.

Review of the Seismic Hazard Zones Map for the Los Gatos Quadrangle (2002) indicates that the site is located within a mapped liquefaction zone. To assess liquefaction potential, we performed liquefaction analyses on two exploratory borings (1-B2 and 1-B3) advanced at the site. We assumed a groundwater level 15 feet below the existing ground surface based on groundwater measurements made during our exploration and as reported in the referenced study prepared by ESG. Additionally, we utilized a PGA of 1.00g and a Mw of 8.0. Our analyses were based on guidelines provided in DMG Special Publication 117A (2008) and methods developed by Youd et al. (NCEER 1998) (2001), Seed (2003), and Boulanger and Idriss (2008).

Based on our analysis, soils encountered at Boring 1-B2 were identified as too dense to liquefy based on review of the blow counts. Soils encountered at Boring 1-B3 were identified as potentially liquefiable in accordance with methods developed by Seed (2003) and Boulanger and Idriss (2008) but were identified as too dense to liquefy based on methods developed by Youd et al. (NCEER 1998) (2001).

As previously mentioned, the majority of the site (with the exception of the southern portion of Building 2) will be excavated to an estimated depth of approximately 20 feet to accommodate the proposed subterranean parking garage. Based on the methodologies outlined above, it is our opinion that the gravel deposits at portions of the site below a depth of approximately 20 feet (with a cumulative thickness of roughly 9 feet) are potentially liquefiable. Additionally, for portions of the site not within the proposed subterranean parking garage, it is our opinion that gravel deposits at portions of the site below a depth of approximately 15 feet (with a cumulative thickness of roughly 14 feet) are also potentially liquefiable. Liquefaction calculations are included in Appendix D.

4.1.4 Liquefaction-Induced Ground Settlement

Our liquefaction analyses indicate that gravel deposits up to 9 feet thick below the bottom of the proposed parking garage (estimated to be 20 feet below grade) may potentially liquefy and result in vertical settlements of approximately 1 inch. Additionally, our liquefaction analyses indicate that gravel deposits up to 14 feet thick for portions of the site not within the proposed parking garage may potentially liquefy and result in vertical settlements of approximately 2 inches.

4.1.5 Lateral Spreading

Lateral spreading can occur in weaker soils on slopes and adjacent to open channels that are subject to strong ground shaking during earthquakes. Based on the relatively flat site topography, variability in density of coarse-grained deposits, and the location of the nearest drainage channel (Los Gatos Creek) roughly 500 feet to the west, it is our opinion that there is a low potential for liquefaction-induced lateral spreading.

4.2 EXISTING FILL

The site is currently occupied by three structures and associated improvements. As such, buried foundation elements and underground utilities may be present on the site.

Existing fills could undergo vertical movement that is not easily characterized and could ultimately be inadequate to effectively support the proposed building loads. In general, undocumented fills should be excavated, and if deemed suitable for reuse, replaced as engineered soil fill. Due to the proposed subterranean parking garage, it is our opinion that the majority of existing fills (if present) will be removed as a result of the garage excavation. However, the extent and quality of existing fills should be evaluated and mitigated during grading activities.

4.3 EXPANSIVE SOILS

Expansive soils shrink and swell as a result of moisture changes. This can cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Atterberg Limits testing performed on samples collected during our field exploration yielded Plasticity Indices (PI) of 19 and 21, indicating a moderate expansive potential of onsite soils.

Successful construction on expansive soils requires special attention during grading. It is imperative to keep exposed soils moist by occasional sprinkling. If the soils dry, it is extremely difficult to remoisturize the soils (because of their clayey nature) without excavation, moisture conditioning, and recompaction.

Conventional grading operations, incorporating fill placement specifications tailored to the expansive characteristics of the soil, and use of a mat foundation are common, generally cost-effective measures to address the expansive potential of the foundation soils. Based upon our initial findings, the effects of expansive soils are expected to pose a low impact when mitigated.

4.4 GROUNDWATER

Groundwater was encountered during our subsurface exploration and during the exploration by ESG on the neighboring property to the east at depths of approximately 18½ to 21 feet bgs. Plate 1.2 of the Seismic Hazard Zone Report for the Los Gatos Quadrangle (2002) indicates historic groundwater highs between approximately 10 to 20 feet below the ground surface.

Based on the above, we recommend using a design groundwater level of 12 feet below existing grade. Fluctuations in groundwater levels should be expected during seasonal changes or over a period of years because of precipitation changes, perched zones, changes in drainage patterns, and irrigation.

4.5 FLOODING

The project Civil Engineer should be consulted on the potential for localized flooding at the subject site. The review should also include a determination of whether the site falls below the 100-year flood plain elevation.

4.6 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS

Considering nearby faults, we provide the 2013 CBC seismic parameters for your use in foundation design. The seismic design parameters presented in the 2013 CBC are based upon the 2012 International Building Code and the ASCE standard "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10) published in 2010. To obtain 2013 CBC seismic parameters, we used the USGS Seismic Design Map online tool to develop ASCE 7-10 seismic design parameters.

TABLE 4.6-1
2013 CBC Seismic Design Parameters

Parameter	Design Value
Site Class	C
Mapped MCE_R spectral response accelerations for short periods, S_S (g)	2.66
Mapped MCE_R spectral response accelerations for 1-second periods, S_1 (g)	1.01
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.30
MCE spectral response accelerations for short periods, S_{MS} (g)	2.66
MCE spectral response accelerations for 1-second periods, S_{M1} (g)	1.31
Design spectral response acceleration at short periods, S_{DS} (g)	1.78
Design spectral response acceleration at 1-second periods, S_{D1} (g)	0.88
Mapped MCE Geometric Mean Peak Ground Acceleration (g)	1.00
Site Coefficient, F_{PGA}	1.00
MCE Geometric Mean Peak Ground Acceleration, PGA_M (g)	1.00
Long period transition-period, T_L	12 sec

MCE_R = Risk-Targeted Maximum Considered Earthquake
MCE = Maximum Considered Earthquake
Latitude: 37.22676; Longitude: -121.97282

4.7 CORROSIVITY CONSIDERATIONS

Two soil samples were collected during the current study and transported under proper chain-of-custody to CERCO Analytical, Inc. for laboratory testing. The samples were tested for redox potential, pH, resistivity, soluble sulfate, and chloride ion concentrations. These tests provide an indication of the corrosion potential of the soil environment on buried concrete structures and metal pipes.

The results are summarized below with the laboratory test results prepared by CERCO Analytical, Inc. contained in Appendix C.

TABLE 4.7-1
Soil Corrosivity Test Results

Sample Number and Depth	Redox Potential (mV)	pH	Resistivity (OHM-CM)	Soluble Sulfate* (mg/kg)	Chloride Ion* (mg/kg)
1-B1 @ 8.5-10 feet	320	7.46	5,300	28	N.D.
1-B3 @ 20-21.5 feet	380	7.47	5,900	32	N.D.

*Results reported on an "as received" basis
N.D – None detected

A corrosion consultant should provide specific design recommendations on corrosion protection for buried metallic lines.

According to the sulfate test results by CERCO, the sulfate ion concentration was reported to range from 28 to 32 mg/kg of water-soluble sulfate (SO_4). The CBC references the American Concrete Institute Manual, ACI 318 (Chapter 4) for concrete requirements. ACI tables provide the following sulfate exposure categories and classes and concrete requirements in contact with soil based upon the exposure risk.

TABLE 4.7-2
Sulfate Exposure Categories and Classes

Sulfate Exposure Category S	Exposure Class	Water- Soluble Sulfate in Soil % by Weight
Not Applicable	S0	$SO_4 < 0.10$
Moderate	S1	$0.10 \leq SO_4 < 0.20$
Severe	S2	$0.20 \leq SO_4 \leq 2.00$
Very Severe	S3	$SO_4 > 2.00$

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TABLE 4.7-3
Requirements for Concrete by Exposure Class

Exposure Class	Max w/cm	Min f'c (psi)	Cement Type			Calcium Chloride Admixture
			ASTM C150	ASTM C595	ASTM C1157	
S0	N/A	2500	No Type restriction	No Type restriction	No Type restriction	No restriction
S1	0.5	4000	II[†‡]	IP(MS), IS(<70), (MS)	MS	No restriction
S2	0.45	4500	V[‡]	IP(HS), IS(<70), (HS)	HS	Not permitted
S3	0.45	4500	V + pozzolan or slag[§]	IP(HS) + pozzolan or slag or IS(<70) (HS) + pozzolan or slag[§]	HS + pozzolan or slag[§]	Not permitted

Notes: † For seawater exposure, other types of portland cements with tricalcium aluminate (C_3A) contents up to 10 percent are permitted if the w/cm does not exceed 0.40.

‡ Other available types of cement such as Type III or Type I are permitted in Exposure Classes S1 or S2 if the C_3A contents are less than 8 or 5 percent, respectively.

§ The amount of the specific source of the pozzolan or slag to be used shall not be less than the amount that has been determined by service record to improve sulfate resistance when used in concrete containing Type V cement. Alternatively, the amount of the specific source of the pozzolan or slag to be used shall not be less than the amount tested in accordance with ASTM C1012 and meeting the criteria in ACI 4.5.1

In accordance with the criteria presented above, the test results are classified in the S0 sulfate exposure class. The minimum concrete strength for this exposure class is specified by the CBC in the table above. As minimum requirements, we recommend that Type II cement be used in foundation concrete for structures at the project site and concrete should incorporate a maximum water cement ratio of 0.5 and a minimum compressive strength of 3,000 psi. It should be noted, however, that the structural engineering design requirements for concrete might result in more stringent concrete specifications.

Testing was not completed for all depths of potential embedment. Once more specifics of the proposed improvements are known, we can provide additional testing and/or guidance regarding the exposure risk for sulfates.

4.8 CONCLUSIONS

From a geologic and geotechnical standpoint, the study area appears to be suitable for the proposed development. The main geologic/geotechnical issues to be addressed at the site are listed below. The recommendations in subsequent sections consider these hazards and concerns.

- Presence of expansive soils.
- Presence of shallow groundwater.
- Potential for liquefaction-induced settlement.

5.0 RECOMMENDATIONS

The recommendations included in this report, along with other sound engineering practices, should be incorporated in the design and construction of the project.

5.1 GRADING

Grading operations should meet the requirements of the Supplemental Recommendations (Appendix E) and should be observed and tested by ENGEO's field representative. ENGEO should be notified a minimum of three days prior to grading in order to coordinate its schedule with the grading contractor.

5.1.1 Demolition and Stripping

Site demolition includes the removal of structures, foundations, and buried structures, including abandoned utilities and septic tanks and their leach fields. Debris and soft compressible soils should be also removed from any location to be graded, from areas to receive fill or structures, or those areas to serve as borrow. The depth of removal of such materials should be determined by the Geotechnical Engineer in the field at the time of grading.

The existing vegetation should be removed from areas to receive fill or improvements, or those areas to serve for borrow. Tree roots should be removed down to a depth of at least 3 feet below existing grade. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with grading operations.

All excavations from demolition and stripping below design grades should be cleaned to a firm undisturbed soil surface determined by the Geotechnical Engineer. This surface should then be scarified, moisture conditioned, and backfilled with compacted engineered fill. The requirements for backfill materials and placement operations are the same as for engineered fill. No loose or uncontrolled backfilling of depressions resulting from demolition or stripping is permitted.

5.1.2 Selection of Materials

With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, organically contaminated materials (soil which contains more than 3 percent organic content by weight), and environmentally impacted soils, we anticipate the site soils are suitable for use as engineered fill. Unsuitable materials and debris, including trees with their root balls, should be removed from the project site.

Subject to approval by the Landscape Architect, organically contaminated soil may be stockpiled in approved areas located outside of the grading limits for future placement within landscape areas. Oversized soil or rock materials (those exceeding two-thirds of the lift thickness or 6 inches in dimension, whichever is less) should be removed from the fill and broken down to meet this requirement or otherwise off-hauled.

The Geotechnical Engineer should be informed when import materials are planned for the site. Import materials should be submitted to, and approved by, the Geotechnical Engineer prior to delivery at the site and should conform to the requirements provided in the Supplemental Recommendations.

5.2 EXISTING FILLS

The site is currently occupied by three structures and associated improvements. As such, buried foundation elements and underground utilities may be present on the site.

Existing fills are considered undocumented and should be subexcavated to expose underlying competent native soils that are approved by the Geotechnical Engineer. If in a fill area, the base of the subexcavations should be processed, moisture conditioned, as needed, and compacted in accordance with the recommendations for engineered fill.

5.3 FILL PLACEMENT

Once a suitable firm base is achieved, the exposed non-yielding native surface should be scarified to a depth of 10 inches, moisture conditioned, and recompacted to provide adequate bonding with the initial lift of fill. All fills should be placed in thin lifts, with the lift thickness not to exceed 10 inches or the depth of penetration of the compaction equipment used, whichever is less.

The following compaction control requirements should be applied to onsite expansive (PI>12) materials:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than 2 percentage points above optimum moisture content.
Required Relative Compaction:	Not less than 92 percent.

The following compaction control requirements should be applied to import or low-expansive (PI<12) soils:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than optimum moisture.
Minimum Relative Compaction:	Not less than 95 percent.

Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material. Additional compaction recommendations may be developed during construction based on materials encountered.

5.4 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. In addition, wet soil conditions are anticipated near the bottom of the parking garage excavation. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather.
2. Mixing with drier materials.
3. Mixing with a lime, lime-flyash, or cement product.
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

Options 3 and 4 should be evaluated and approved by ENGEO prior to implementation.

5.5 GRADED SLOPES

In general, graded slopes should be no steeper than 2:1 (horizontal:vertical). All fill slopes should be adequately keyed into firm materials unaffected by shrinkage cracks. If a cut or cut-fill transition occurs within a graded slope, we recommend that it be overexcavated and reconstructed as an engineered fill slope.

5.6 MONITORING AND TESTING

It is important that all site preparations for site grading be done under the observation of the Geotechnical Engineer's field representative. The Geotechnical Engineer's field representative should observe all graded area preparation, including demolition and stripping, following the recommendations contained herein and in the Supplemental Recommendations.

The final grading and foundation plans should be submitted to the Geotechnical Engineer for review.

5.7 FOUNDATION DESIGN

Although the preliminary structural concept and foundation loads have not yet been developed, based on our experience with similar projects, we anticipate the proposed podium structure may be supported on a stiff structural mat foundation.

As previously mentioned, the southern portion of Building 2 will be located outside of the footprint of the subterranean parking garage. The portion of Building 2 that extends outside of the parking garage should be structurally designed to cantilever or span the distance unsupported. If the distance or loading conditions are too great, additional support from drilled piers with interconnected grade beams may be required. We can provide supplemental recommendations if needed.

The proposed podium building will have two levels of subterranean parking that will extend below the design groundwater level. The structure will need to be designed to resist hydrostatic uplift pressures based on the design groundwater level.

5.7.1 Potential Total and Differential Settlement

Assuming the subterranean parking garage extends at least a distance of 20 feet below grade, we recommend that the foundation design consider 1 inch of total and ½ inch of differential settlement associated with liquefaction-induced settlement. The differential settlement may be assumed to occur over a distance of 30 feet or between adjacent column supports, whichever is closer.

5.7.2 Buoyancy Impacts

The garage will be below the 12-foot design groundwater level and will be subject to buoyancy impacts. The foundation should be designed to resist hydrostatic uplift pressures due to the design groundwater level of 12 feet below existing grade. Uplift resistance can be provided by the weight of the foundation elements and the dead loads of the building. The structural engineer should evaluate the buoyancy uplift on the structure and determine if additional resistance is necessary. Viable alternatives for added uplift resistance include hold-down piers or anchors. These can be designed as active or passive systems and we can provide more details as necessary.

5.7.3 Building Pad Treatment

We recommend the subgrade consist of 12 inches of uniform engineered fill. This can be accomplished by subexcavating to pad subgrade followed by scarifying, mixing, moisture conditioning, and compacting the exposed surface to a depth of approximately 12 inches.

If loose/compressible soils are encountered, they should be removed and replaced with compacted engineered fill. Geotextile stabilization fabric may also be recommended in the field.

Considering the shallow groundwater conditions encountered at the site, the exposed subgrade may be near saturation. In addition, the building pad will be susceptible to disturbance under construction equipment loads. The contractor should limit the use of rubber-tired equipment on the subgrade to reduce potential for creation of unstable areas. The contractor should also consider chemical treatment of the subgrade, especially if construction will occur during wet weather. Alternatively, a working pad can be constructed to assist in protecting the subgrade soils.

5.7.4 Structural Mat Foundation

The proposed building can be supported on a conventional mat foundation. The rigid mat should be designed to impose a maximum allowable bearing capacity of 4,000 pounds per square foot (psf) for dead plus long-term live loads. These values may be increased by one-third when considering transient loads, such as wind or seismic. A modulus of subgrade reaction of 75 psi per inch of deflection can be used for engineered fill or native soil. This value represents the modulus of subgrade reaction for a 1 square foot bearing plate.

Resistance to lateral loads may be provided by frictional resistance between the foundation concrete and the subgrade soils and by passive earth pressure acting against the side of the foundation. A coefficient of friction of 0.35 can be used between concrete and the subgrade. Passive pressures can be taken as equivalent to the pressure developed by a fluid having a weight of 300 pounds per cubic foot (pcf).

Localized liquefaction within gravel deposits located below the mat foundation may result in a reduction in bearing capacity and foundation subgrade soil stiffness. To model this condition, we recommend assuming that the localized bearing capacity and stiffness are reduced to zero. This can be modeled by designing the mat foundation to withstand an edge cantilever distance of 6 feet and an interior span distance of 15 feet.

The concrete slabs should be waterproofed, as discussed in a subsequent section. A double-slab drainage system may also be considered to reduce the chance of moisture or water ponding within the lower garage level.

The subgrade material under a mat foundation should be uniform and the mat should be placed neat against the undisturbed soil. The pad subgrade should not be allowed to dry before placing concrete. The pad subgrade should be checked by a representative of ENGEO prior to concrete placement for compliance with these moisture requirements and to confirm the adequacy of the bearing soil.

5.8 BUILDING RETAINING WALLS

We anticipate the underground parking structure will include below-grade retaining walls approximately 20 feet high constructed on a structural mat foundation or continuous footings.

5.8.1 Design Recommendations

The building retaining walls should be designed to resist lateral earth pressures from natural materials and/or backfill and from any surcharge loads. Provided that adequate drainage is included as recommended below, the restrained walls may be designed using an at-rest equivalent fluid pressure of 60 pcf. The design should account for one-half of any vertical surcharge loads applied as a uniform lateral load to the top 10 feet of the wall.

The building walls should have drainage facilities above the design groundwater depth of 12 feet below existing grade to reduce the potential for build-up of hydrostatic pressures. If the walls are not designed with adequate drainage, we recommend adding an additional equivalent fluid pressure of 40 pcf. The wall design should include the additional 40 pcf hydrostatic pressure for depths greater than the design depth to groundwater of 12 feet below ground surface.

We recommend the seismic performance of the basement retaining walls be evaluated using an active equivalent fluid weight of 40 pcf for drained conditions and an active equivalent fluid weight of 80 pcf for undrained conditions, and a seismic increment of 20 pcf, in accordance with Lew, et al. (2010). This evaluation should be separate from the static design using at-rest earth pressures.

Passive pressures acting on foundations may be assumed as 300 pounds per cubic foot (pcf). A coefficient of friction of 0.35 can be used between concrete and the subgrade.

Basement retaining walls should be waterproofed as discussed in a subsequent section.

5.8.2 Wall Drainage

Design details for draining the basement walls above the groundwater level should be determined during the design process. A sump system may be needed for drainage at this elevation unless the storm drain system will allow for gravity connection.

Construct either graded rock drains or geosynthetic drainage composites behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives:

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-1.025) placed directly behind the wall, or

2. A minimum 12-inch-thick layer of washed, crushed rock. Envelop rock in a minimum 6-ounce, nonwoven geotextile filter fabric.

For both types of rock drains:

1. Place the rock drain directly behind the walls of the structure.

2. Extend rock drains from a depth of 12 feet below the ground surface to within 12 inches of the top of the wall.

3. Place a minimum of 4-inch-diameter perforated pipe at the base of the drain material, inside the rock drain and fabric, with perforations placed down.

4. Place pipe at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

ENGEO should review and approve geosynthetic composite drainage systems prior to use.

5.8.3 Backfill

Backfill behind retaining walls should be placed and compacted in accordance with fill placement recommendations. Use light compaction equipment within 5 feet of the wall face. If moderate to heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement. Alternatively, the wall design can incorporate additional surcharge loading to allow moderate to heavy equipment.

5.9 WATERPROOFING

Permanent dewatering is not recommended and the mat foundation or concrete slabs and basement walls should be waterproofed and designed to resist hydrostatic and/or uplift pressures. The waterproofing should be designed by a consultant that specializes in permanent waterproofing construction. Waterstops should be placed at all construction joints.

5.10 SITE RETAINING WALLS

This section is intended for walls, if any, located outside of the main building that are needed for grades separations or landscaping. Unrestrained, drained retaining walls constructed on level ground and up to 6 feet in height may be designed using active equivalent fluid pressures as follows.

TABLE 5.10-1
Active Equivalent Fluid Pressures

Backfill Slope Condition (horizontal:vertical)	Active Pressure (pounds per cubic foot)
Level	40
3:1	50
2:1	60

Site retaining walls should be designed using an allowable bearing capacity of 2,500 psf. Site retaining wall footings should be founded at least 12 inches below adjacent grade. Passive pressures acting on foundations may be assumed as 250 pcf provided the area in front of the retaining wall is level for a distance of at least 10 feet or three times the depth of the foundation, whichever is greater. Unless the surface in front of the wall is confined by a slab or pavement, the upper one foot of soil should be neglected when calculating passive resistance. A coefficient of friction of 0.35 can be used between concrete foundation and the subgrade. Appropriate safety factors against overturning and sliding should be incorporated into the design calculations.

Wall drainage should be included as discussed in a previous section.

All backfill should be placed in accordance with the recommendations provided above for engineered fill. Light equipment should be used during backfill compaction to reduce possible overstressing of the walls. The foundation details and structural calculations for retaining walls should be submitted for review.

5.11 TEMPORARY EXCAVATIONS

The Contractor should be familiar with applicable local, state, and federal regulations, including the current Occupational Safety and Health Administration (OSHA) Excavation and Trench Safety Standards. It is the responsibility of the Contractor to provide stable, safe trench and construction slope conditions and to follow OSHA safety requirements. Since excavation procedures may be dangerous, it is also the responsibility of the Contractor to provide a trained "competent person" as defined by OSHA to supervise all excavation operations, ensure that all personnel are working in safe conditions and have thorough knowledge of OSHA excavation safety requirements.

Based on the soil data, excavations up to approximately 20 feet deep may generally consider classification of Type C soil in Cal OSHA shoring, sloping, and benching design (i.e., maximum 1½:1 temporary cut slopes). The Geotechnical Engineer should be present during the excavation of site soils to provide geotechnical recommendations as necessary and identify variations in soil conditions as appropriate.

5.12 TEMPORARY SHORING

We anticipate excavations up to 20 feet deep for the parking garage construction. At this time, we anticipate a cantilevered temporary shoring system consisting of drilled or driven soldier piles with lagging will be utilized. If a cantilevered shoring system is not feasible, we can provide supplemental recommendations for a restrained system.

Applicable loading, including surcharges due to traffic, buildings, stockpiles, construction equipment, etc. should be incorporated into shoring design when the surcharge loading is situated above a 1:1 line of projection extending up the bottom of wall. A uniform, horizontal surcharge loading (in units of pounds per square foot) of 50 percent of the vertical surcharge load should be assumed to act over the upper 10 feet of the wall. Appropriate safety factors against overturning and sliding should also be incorporated into the design calculations.

We anticipate that the final temporary shoring design will be based on the contractor's means and methods of construction, including equipment and available shoring materials, as well as other general conditions defined by the project team. Recommendations for a temporary soldier pile and lagging shoring system are provided below.

We recommend the following design parameters be used for cantilevered walls. As noted above, braced or tieback walls will require additional recommendations.

TABLE 5.12-1
Temporary Soldier Pile and Lagging Shoring Design Parameters

Temporary Shoring Design Element	Design Parameter
Active Earth Pressure:	40 pcf (Level backfill conditions) Active earth pressures should be used where existing buildings and critical utilities are situated outside a 1:1 line of projection extending up from the bottom of the wall
At-Rest Earth Pressure:	60 pcf (Level backfill conditions) At-rest earth pressures should be used where existing buildings and critical utilities are situated within a 1:1 line of projection extending up from the bottom of the wall
Passive Earth Pressure:	300 pcf for soil conditions and 500 pcf for bedrock conditions (anticipated below El. 307, approximate), acting on three times the pier diameter provided the soldier pile is backfilled with structural concrete, if drilled. This value may be increased by ⅓ when considering seismic loads.

5.13 TEMPORARY DEWATERING

Based on the anticipated depths of approximately 20 feet for the planned excavation and considering groundwater levels encountered during our field exploration and a design groundwater level of 12 feet, groundwater may be encountered above the bottom of the excavation. Temporary dewatering during construction may be necessary. Assessment of dewatering should be made prior to excavation to determine the level of groundwater control and dewatering necessary to address long-term conditions for the depressed portions of the structure at this site.

Temporary dewatering during construction may be necessary to keep the excavation and working areas reasonably dry. If necessary, dewatering should be performed in a manner such that water levels are maintained not less than 2 feet below the bottom of excavation prior to and continuously during shoring and foundation installation. As the excavations progress, it may be necessary to dewater the soils ahead of the excavation, such as by continuous pumping from sumps, to control the tendency for the bottom of the excavation to heave under hydrostatic pressures and to reduce inflow of water or soil from beneath temporary shoring.

The selection of equipment and methods should be determined by the dewatering designer/contractor. The dewatering system implemented should be selected so as to have minimal impact on the groundwater level surrounding the proposed excavation.

5.14 SECONDARY SLAB-ON-GRADE CONSTRUCTION

This section provides guidelines for secondary slabs such as exterior walkways, steps, and sidewalks. Secondary slabs-on-grade should be constructed structurally independent of the foundation system. This allows slab movement to occur with a reduced potential for foundation distress. Where secondary slab-on-grade construction is anticipated, care must be exercised in attaining a near-saturation condition of the subgrade soil before concrete placement.

Secondary slabs-on-grade should be designed specifically for their intended use and loading requirements. Cracking of conventional slabs should be expected as a result of concrete shrinkage and the expansive soils at the site. Slabs-on-grade should be reinforced for control of cracking, and frequent control joints should be provided to control the cracking. Such reinforcement should be designed by the Structural Engineer. In our experience, welded wire mesh may not be sufficient to control slab cracking. There are numerous measures that can be implemented to improve the performance of exterior slabs. We would be pleased to consult with you in this regard if desired.

Secondary slabs-on-grade not subject to vehicular loads should have a minimum thickness of 4 inches and be underlain by at least 4 inches of clean crushed rock or gravel. Secondary slabs-on-grade that are subject to vehicular loads should have a minimum thickness of 4 inches and be underlain by at least 6 inches of clean crushed rock or gravel. Exterior slabs should be constructed with thickened edges extending at least beneath the crushed rock or gravel into compacted soil to reduce water infiltration. Slabs should slope away from the buildings at a slope of at least 2 percent to prevent water from flowing toward the building.

5.15 PRELIMINARY PAVEMENT DESIGN

Preliminary pavement design is provided based on assumed Traffic Index and subgrade resistance values (R-value). The Traffic Index should be determined by the Civil Engineer or appropriate public agency. The sections provided below should be revised, if applicable, based on R-value tests performed on samples of actual subgrade materials recovered at the time of grading.

Based on the referenced plans prepared by Architectural Technologies, portions of the entry driveway and circular at grade parking area are underlain by the subterranean parking structure while some portions extend outside of the limits of the parking structure. As such, minor settlement of the parking structure may cause minor cracking of pavements in locations that straddle these transition zones. If possible, at-grade improvements should be located such that they are situated entirely within or outside of the limits of the parking garage. We can provide supplemental recommendations at a later date if relocating of surface improvements can't be achieved.

5.15.1 Flexible Pavement

Based on our field exploration, we estimate that site soil will have a resistance (R-value) value of 5. The following preliminary pavement sections have been determined based on an assumed R-value of 5 according to the method contained in the Highway Design Manual by CALTRANS.

TABLE 5.15.1-1
Preliminary Flexible Pavement Design

Traffic Index (TI)	R-Value of 5 (untreated subgrade)	
	AC (inches)	AB (inches)
4.0	2.5	8.0
5.0	3.0	10.0
6.0	3.5	13.0
7.0	4.0	16.0

Notes: AC is asphalt concrete
AB is aggregate base Class 2 Material with minimum R = 78

5.15.2 Rigid Pavements

We developed recommended pavement sections using the Portland Cement Association Thickness Design for Concrete Highway and Street Pavements manual (1995) based on the assumed subgrade soil type. We recommend the following minimum design sections for rigid pavements:

- Use a minimum section of 8 inches of Portland Cement concrete over 8 inches of Class 2 aggregate base.
- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

5.15.3 Pavement Subgrade Preparation

Pavement construction and all materials (hot mix asphalt and aggregate base) should comply with the requirements of the Standard Specifications of the State of California Division of Highways, Town of Los Gatos requirements and the following minimum requirements.

- All pavement subgrades should be scarified to a depth of 10 to 12 inches below finished subgrade elevation, moisture conditioned to at least 2 percentage points above optimum moisture content, and compacted to at least 95 percent relative compaction and in accordance with Town of Los Gatos requirements.
- Subgrade soils should be in a stable, non-pumping condition at the time aggregate baserock materials are placed and compacted. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor and Geotechnical Engineer.

- Aggregate baserock materials should meet current Caltrans specifications for Class 2 aggregate baserock and should be compacted to at least 95 percent of maximum dry density at a moisture content of at least optimum. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented after placement and compaction of the aggregate base. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor and Geotechnical Engineer.

- Adequate provisions must be made such that the subgrade soils and aggregate baserock materials are not allowed to become saturated.

- All concrete curbs separating pavement and irrigated landscaped areas should extend into the subgrade and below the bottom of adjacent aggregate baserock materials. An undercurb drain could also be considered to help collect and transport subsurface seepage.

5.16 DRAINAGE

Perimeter grades should be positively sloped at all times to provide for rapid removal of surface water runoff away from the foundation systems and to prevent ponding of water under foundations or seepage toward the foundation systems at any time during or after construction. Ponded water may cause undesirable soil swell and loss of strength. As a minimum requirement, finished grades should have slopes of at least 5 percent within 10 feet from the exterior walls and at right angles to allow surface water to drain positively away from the structure. For paved areas, the slope gradient can be reduced to 2 percent.

All surface water should be collected and discharged into outlets approved by the Civil Engineer. Landscape mounds must not interfere with this requirement.

All roof stormwater should be collected and directed to downspouts. Stormwater from roof downspouts should not be allowed to discharge directly onto the ground surface in close proximity to the foundation system, such as via spashblocks. Rather, stormwater from roof downspouts should be directed to a solid pipe that discharges into the street or to an outlet approved by the Civil Engineer. If this is not acceptable, we recommend downspouts discharge at least 5 feet away from foundations. Alternatively, engineered stormwater systems can be developed under the guidance of ENGEO.

5.17 STORMWATER INFILTRATION AND BIORETENTION AREAS

Based on the anticipated fines content and laboratory test results, the near-surface site soils are expected to have low permeability values to handle stormwater infiltration. Post-construction BMPs should not rely on infiltration; rather, we recommend BMPs receive subdrains that discharge treated stormwater into the planned storm drain system.

If possible, we recommend bioswales/bioretention areas and other BMPs be planned a minimum of 5 feet away from structural site improvements. Where this is not practical, bioretention areas located within 5 feet of structural onsite or offsite improvements can either:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or

2. Incorporate filter material compacted to between 85 and 90 percent relative compaction (ASTM D1557, latest edition).

In addition, one of the following options should be followed:

1. Bioretention design should incorporate a waterproofing system lining the bioswale excavation and a subdrain, or other storm drain system, to collect and convey water to an approved outlet. The waterproofing system should cover the bioretention area excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.

2. Alternatively, and with increased risk of movement of adjacent improvements, if minor infiltration is desired, the perimeter of the bioretention areas should be lined with an HDPE tree root barrier that extends at least 1 to 2 feet below the bottom of the bioretention area.

In addition, site improvements located adjacent to bioretention areas that are underlain by base rock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement. Where adjacent site improvements include design elements that will experience lateral loads (such as from impact or traffic patterns), additional design considerations may be required.

Given the nature of bioretention systems and possible proximity to improvements, we recommend ENGEO consult further with you as needed, review design plans, and provide testing and observation services during the installation of linings, compaction of the filter material, and connection of designed drains (if implemented).

It should be noted that the contractor is responsible for conducting all excavation and shoring in a manner that does not cause damage to adjacent improvements during construction and future maintenance of the bioretention areas. As with any excavation adjacent to improvements, the contractor should minimize the exposure time such that the improvements are not detrimentally impacted.

5.18 REQUIREMENTS FOR LANDSCAPING IRRIGATION

The geotechnical foundation design parameters contained in this report have considered the swelling potential of some of the site soils; however, it is important to recognize that swell in excess of that anticipated is possible under adverse drainage or irrigation conditions. Therefore, planted areas should be avoided immediately adjacent to the buildings. If planting adjacent to the structure is desired, the use of watertight planter boxes with controlled discharge or the use of plants that require very little moisture is recommended.

Sprinkler systems should not be installed where they may cause ponding or saturation of foundation soils within 5 feet from walls. Such ponding or saturation could result in undesirable soil swell, loss of compaction and consequent foundation and slab movements. Irrigation of landscaped areas should be strictly limited to that necessary to sustain vegetation. The Landscape Architect and prospective owners should be informed of the surface drainage and irrigation requirements included in this report.

5.19 UTILITIES

It is recommended that utility trench backfilling be done under the observation of a Geotechnical Engineer. Ideally, pipe zone backfill (i.e. material beneath and immediately surrounding the pipe) should consist of native material less than ¾ inch in maximum dimension compacted in accordance with recommendations provided above for engineered fill. Trench zone backfill (i.e. material placed between the pipe zone backfill and the ground surface) should also consist of native soil compacted in accordance with recommendations for engineered fill. Controlled density fill is also suitable for pipe zone and trench zone backfill.

If required by local agencies, where import material is used for pipe zone backfill, we recommend it consist of quarry fines, fine- to medium-grained sand, or a well-graded mixture of sand and gravel and that this material not be used within 2 feet of finish subgrades. This material should be compacted to at least 90 percent relative compaction at a moisture content of not less than optimum.

In general, uniformly graded gravel should not be used for pipe or trench zone backfill due to the potential for migration of soil into the relatively large void spaces present in this type of material and for movement of water along trenches backfilled with this type of material. If uniformly graded gravel is used, we recommend that it be encapsulated in 6-ounce filter fabric. Providing outlet locations into manholes or catch basins for water collected in granular trench backfill should also be considered.

All utility trenches entering the buildings and paved areas should be provided with an impervious seal where the trenches pass under or through the building perimeter or curb lines. The impervious plug should extend at least 3 feet to both sides of the crossing and should be placed below, around, and above the utility pipe such that it is entirely in contact with the trench walls and pipe. This is to prevent surface water percolation into the import sand or gravel pipe zone backfill under foundations and pavements where such water would remain trapped in a perched condition.

Care should be exercised where utility trenches are located beside foundation areas. Utility trenches constructed parallel to foundations should be located entirely above a plane extending down from the lower edge of the footing at an angle of 45 degrees. Utility companies and Landscape Architects should be made aware of this information.

Utility trenches in areas to be paved should be constructed in accordance with the Town of Los Gatos requirements or approved alternatives. Compaction of backfill by jetting should not be allowed at this site. If there appears to be a conflict between the Town or other Agency requirements and the recommendations contained in this report, this should be brought to the Owner's attention for resolution prior to submitting bids.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to developers, contractors, buyers, architects, engineers, and designers for the project so that the necessary steps can be taken by the contractors and subcontractors to carry out such recommendations in the field. The conclusions and recommendations contained in this report are solely professional opinions.

The professional staff of ENGEO Incorporated strives to perform its services in a proper and professional manner with reasonable care and competence but is not infallible. There are risks of earth movement and property damages inherent in land development. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of preparation of ENGEO's documents of service. This document must not be subject to unauthorized reuse, that is, reuse without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time. Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims, including, but not limited to claims arising from or resulting from the performance of such services by other persons or entities, and any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

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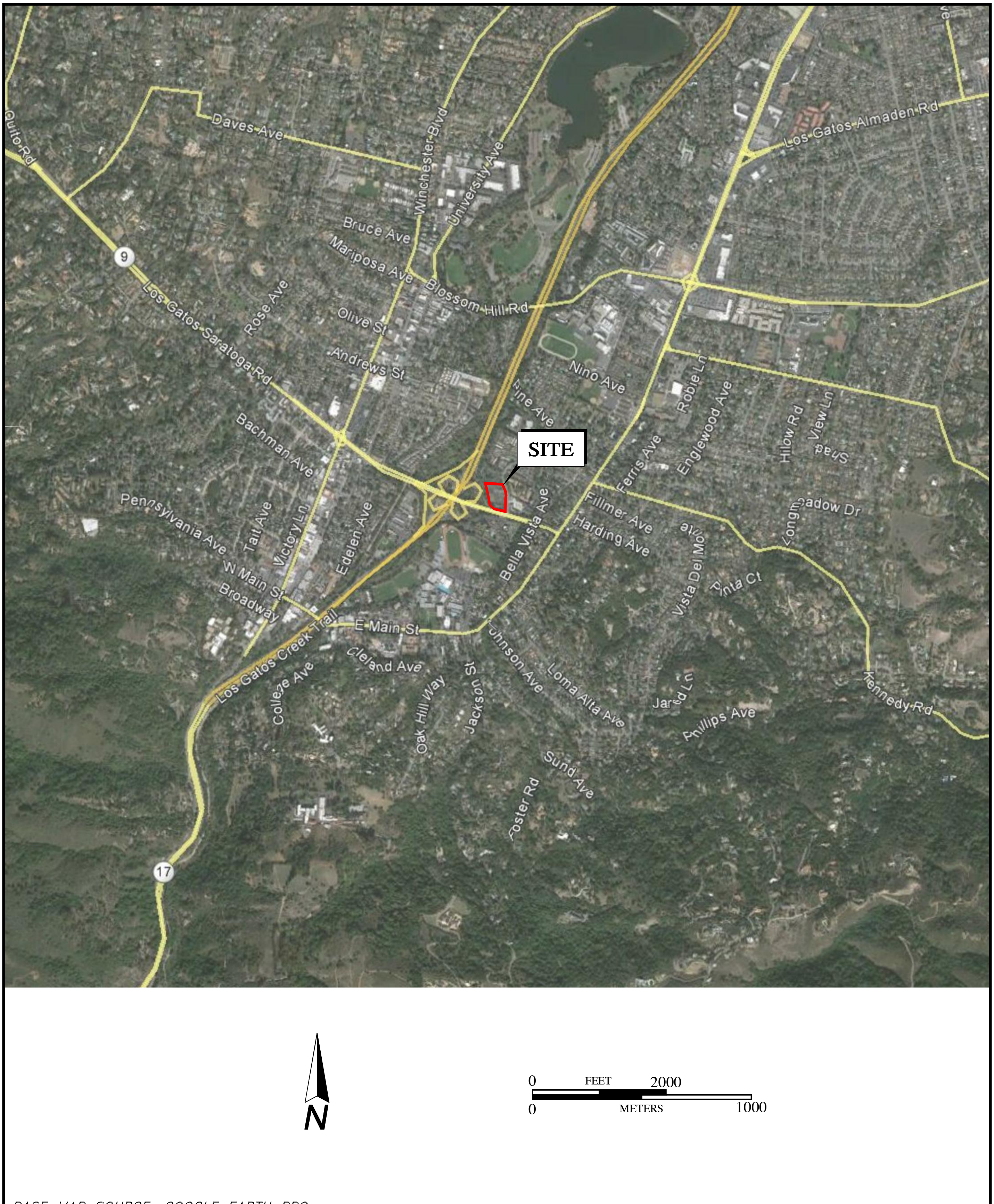
FIGURES



BASE MAP SOURCE: GOOGLE EARTH PRO

ENGEO
Expect Excellence

VICINITY MAP
401 ALBERTO WAY
LOS GATOS, CALIFORNIA

PROJECT NO.: 12175.000.000	FIGURE NO. 1
SCALE: AS SHOWN	
DRAWN BY: LL	CHECKED BY: BB

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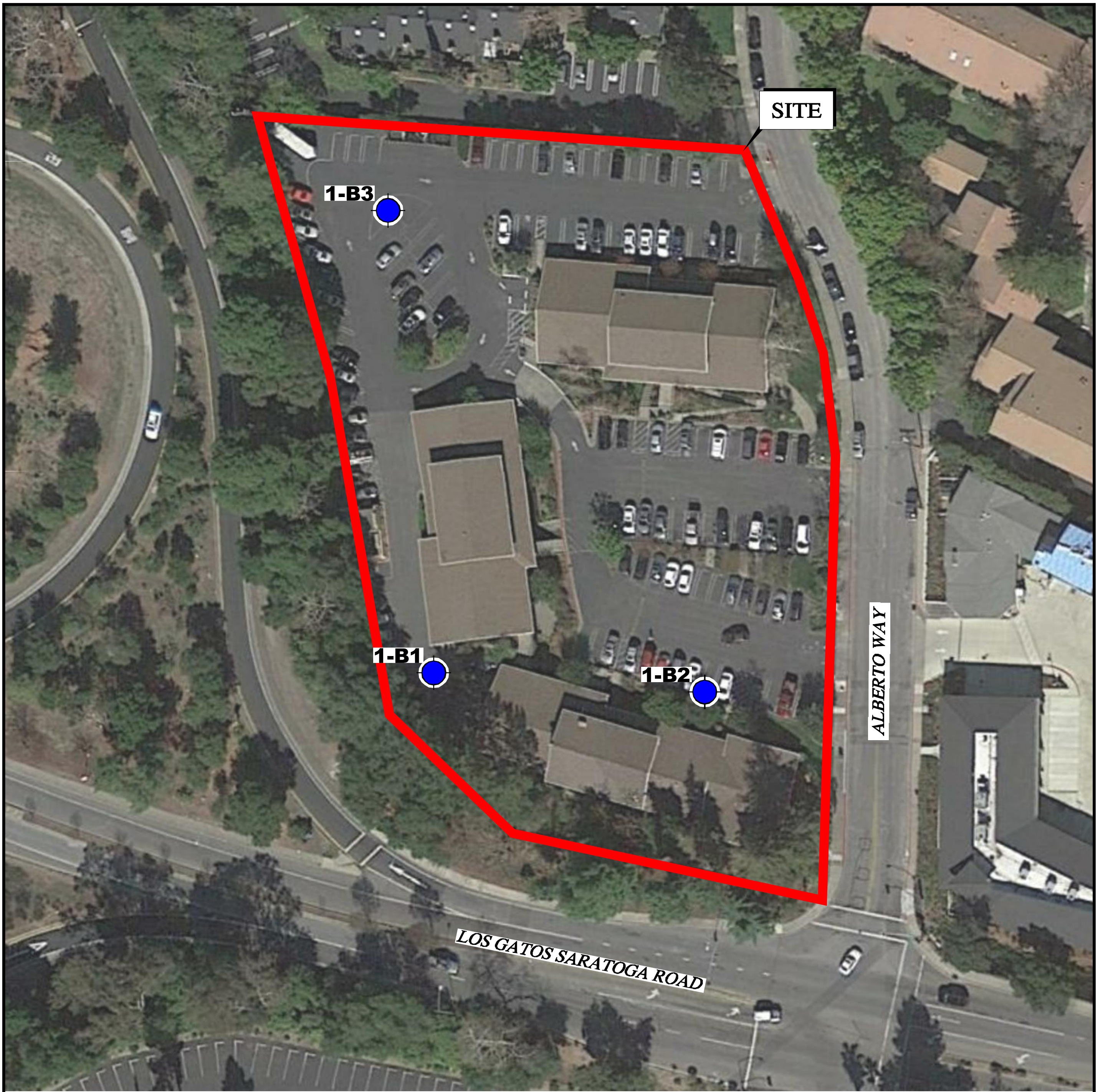
ORIGINAL FIGURE PRINTED IN COLOR



N

0 FEET 80
0 METERS 40

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-B3 BORING

BASE MAP SOURCE: GOOGLE EARTH PRO

ENGEO Expect Excellence	SITE MAP 401 ALBERTO WAY LOS GATOS, CALIFORNIA	PROJECT NO.: 12175.000.000 SCALE: AS SHOWN DRAWN BY: LL CHECKED BY: BB	FIGURE NO. 2

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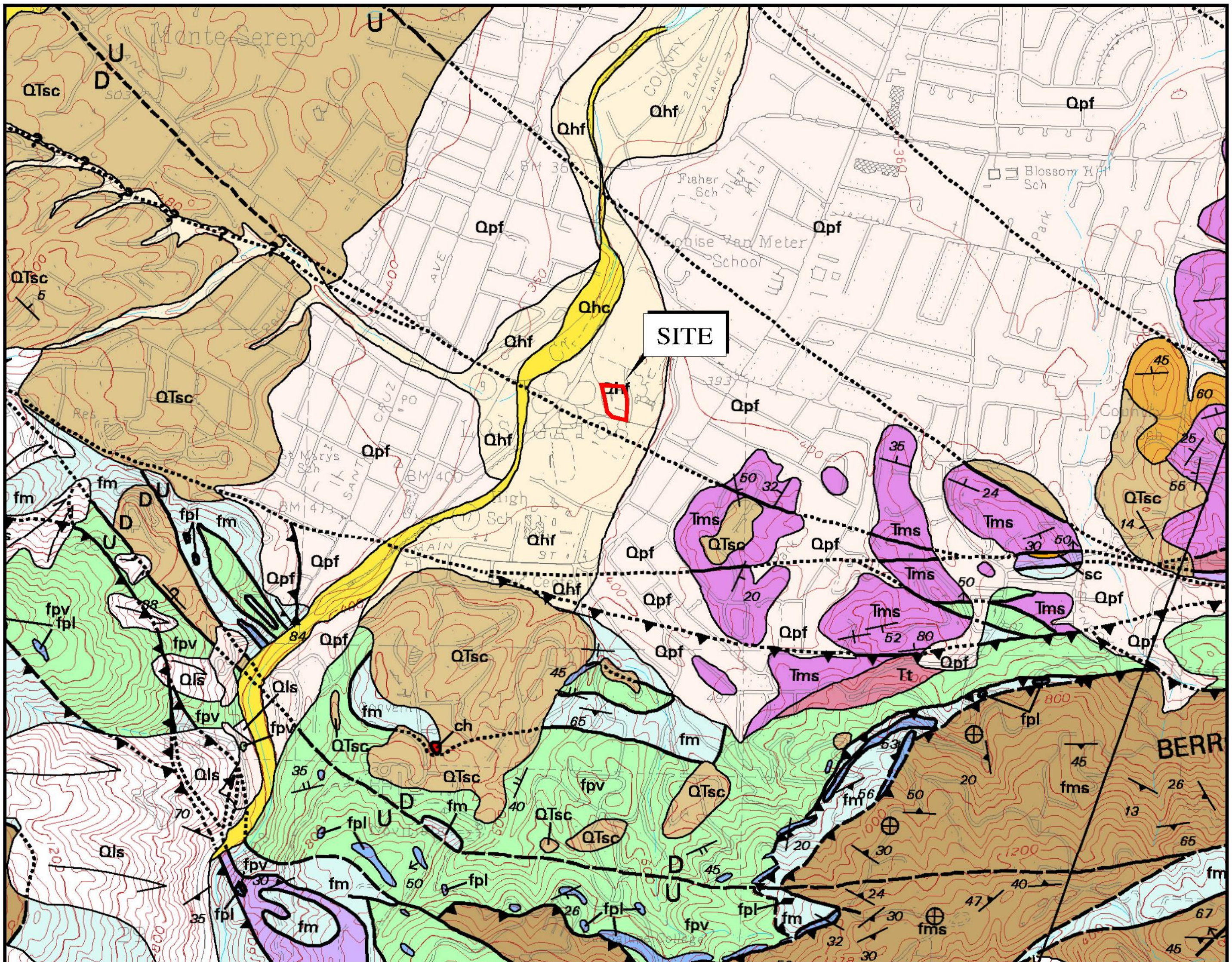
ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL. SAWTEETH ARE ON UPPER PLATE OF LOW ANGLE THRUST FAULT.

DIRECTION OF LANDSLIDE MOVEMENT

STRIKE AND DIP OF STRATA

INCLINED VERTICAL OVERTURNED

Qls	LANDSLIDE DEPOSITS, UNDIVIDED
Qhc	STREAM CHANNEL DEPOSITS (HOLOCENE)
Qhf	ALLUVIAL FAN DEPOSITS (HOLOCENE)
Qpf	ALLUVIAL FAN DEPOSTS (PLEISTOCENE)
QTsc	SANTA CLARA FORMATION
sc	SILICA-CARBONATE ROCK
Tms	MONTEREY SHALE
Tt	TEMBLOR SANDSTONE
Jos	SERPENTINIZED ULTRAMAFIC ROCKS
fpl	FORAMINFERAL LIMESTONE
fpv	VOLCANIC ROCKS
fms	SANDSTONE
ch	CHERT BLOCKS

N

0 FEET 2000

0 METERS 1000

BASE MAP SOURCE: MCLAUGHLIN, 2001

ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP

401 ALBERTO WAY

LOS GATOS, CALIFORNIA

PROJECT NO.:	12175.000.000
SCALE:	AS SHOWN
DRAWN BY: LL	CHECKED BY: BB
FIGURE NO.	3

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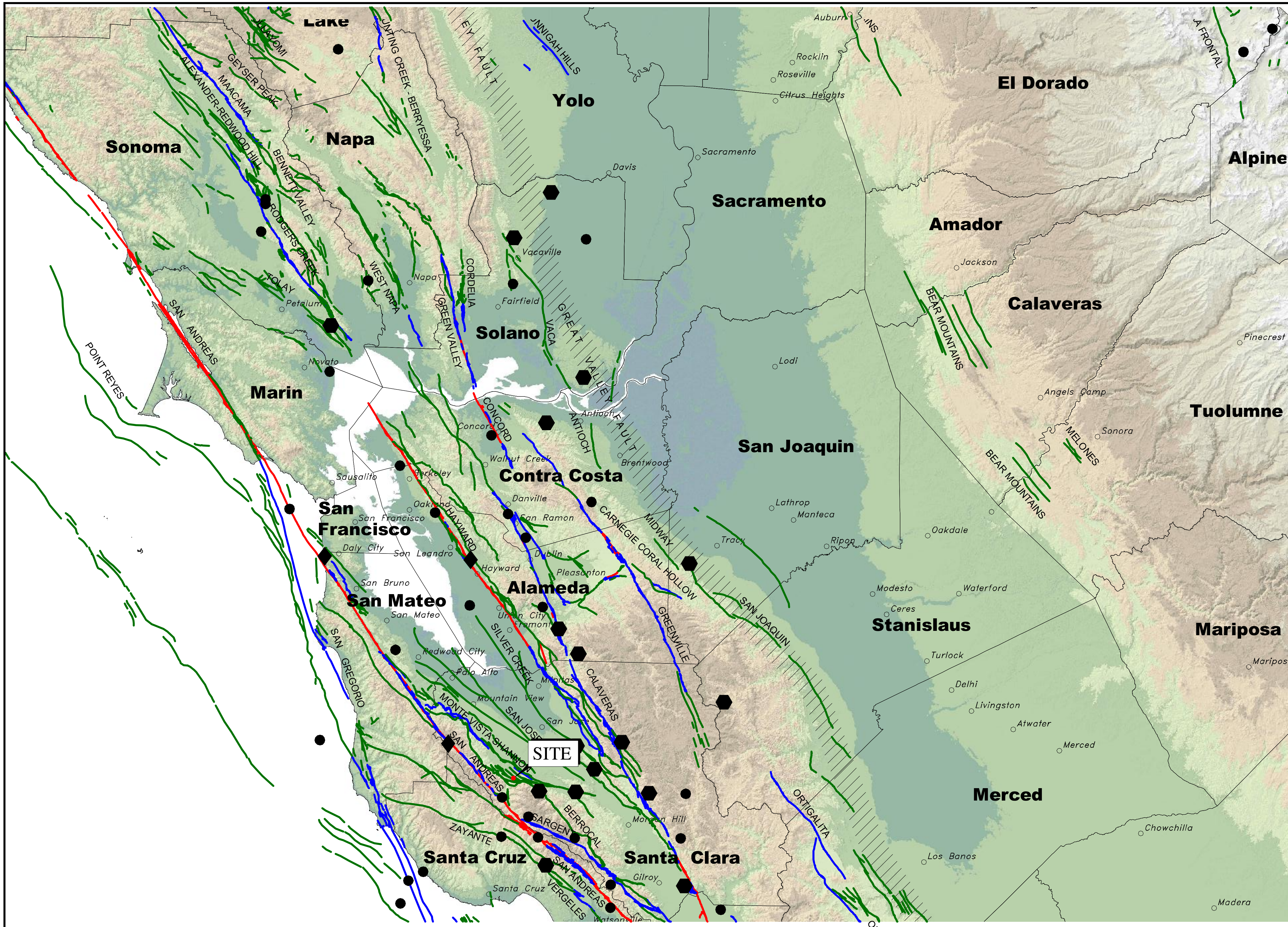
ORIGINAL FIGURE PRINTED IN COLOR



BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

0 MILES 15
0 KILOMETERS 30

EXPLANATION

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6
- HISTORIC FAULT
- HOLOCENE FAULT
- QUATERNARY FAULT
- HISTORIC BLIND THRUST FAULT ZONE

ENGEO Expect Excellence	REGIONAL FAULTING AND SEISMICITY 401 ALBERTO WAY LOS GATOS, CALIFORNIA	PROJECT NO.: 12175.000.000 SCALE: AS SHOWN DRAWN BY: LL CHECKED BY: BB	FIGURE NO. 4

ORIGINAL FIGURE PRINTED IN COLOR

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APPENDIX A

Boring Logs
(ENGEO, 2015)

APPENDIX A

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures
			GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures
			GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures
			SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures
			SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity
			CL - Inorganic clay with low to medium plasticity
			OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity
			CH - Fat clay with high plasticity
			OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE: 200, 40, 10, 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4 ", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

LINE TYPES

- Solid - Layer Break
- Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

ENGEO Expect Excellence

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

ENGEO INCORPORATED

LOG OF BORING 1-B1

Geotechnical Exploration
401 Alberto Way
Los Gatos, California
12175.000.000

DATE DRILLED: 6/27/2015
HOLE DEPTH: Approx. 15 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (): Approx. 341½ ft.

LOGGED / REVIEWED BY: I. McCreery / PCG
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
0			3.5 inches AC over 4 inches AB												
	1		CLAYEY SAND WITH GRAVEL (SC), dark reddish brown, medium dense, slightly moist			17	37	18	19		9.4	98.7	1290.8		UU
5	2														
10	3		CLAYEY SAND WITH GRAVEL (SC), olive brown, medium dense, slightly moist			19									
	4		CLAYEY SAND WITH GRAVEL (SC), brown and gray, dense, slightly moist			34									
15			Total depth approxiamtely 15 feet bgs. Groundwater not encountered during drilling. Backfilled with grout.												

ENGEO INCORPORATED

LOG OF BORING 1-B2

Geotechnical Exploration
401 Alberto Way
Los Gatos, California
12175.000.000

DATE DRILLED: 6/27/2015
HOLE DEPTH: Approx. 40½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (): Approx. 340 ft.

LOGGED / REVIEWED BY: I. McCreery / PCG
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3 inches AC over 4 inches AB												
	1		CLAYEY SAND WITH GRAVEL (SC), dark reddish brown, medium dense, slightly moist			17					9.9	102.4		1.35	UC
5	2		CLAYEY SAND WITH GRAVEL (SC), dark reddish brown, medium dense, slightly moist			16					7.5	103.4			
			CLAYEY SAND WITH GRAVEL (SC), dark reddish brown, medium dense, slightly moist			15									
10	3					50 for 6 in.									
			POORL GRADED SAND WITH GRAVEL (SP), gray and dark reddish brown, very dense, slightly moist												
15	4, 5		CLAYEY SAND WITH GRAVEL (SC), dark reddish brown, dense, moist			44	39	18	21						
20	6		CLAYEY SAND WITH GRAVEL (SC), olive brown, dense, moist		▽	98				12					
25	7		POORLY GRADED GRAVEL WITH CLAY AND SAND (GP-GC), yellowish brown and red, very dense, moist, contains some sandstone rock fragments												

ENGEO INCORPORATED

LOG OF BORING 1-B2

Geotechnical Exploration
401 Alberto Way
Los Gatos, California
12175.000.000

DATE DRILLED: 6/27/2015
HOLE DEPTH: Approx. 40½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (): Approx. 340 ft.

LOGGED / REVIEWED BY: I. McCreery / PCG
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	8		POORLY GRADED GRAVEL (GP-GC), blue gray, very dense, wet			54									
30	9		POORLY GRADED GRAVEL (GP-GC), blue gray, very dense, wet			61									
35	10, 11		SHALE, very dark brown, weak (R2), very closely fractured, damp			101 for 7 in.									
40	12					50 for 6 in.									
			Total depth approximately 40.5 feet bgs. Groundwater encountered at approximately 21 feet during drilling. Backfilled with grout.												

ENGEO INCORPORATED

LOG OF BORING 1-B3

Geotechnical Exploration
401 Alberto Way
Los Gatos, California
12175.000.000

DATE DRILLED: 6/27/2015
HOLE DEPTH: Approx. 34¾ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (): Approx. 338½ ft.

LOGGED / REVIEWED BY: I. McCreery / PCG
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			4.5 inches AC over 1 inch AB												
	1		CLAYEY SAND (SC-CL), dark reddish brown, medium dense, slightly moist			11				47	12.6	102.5		1.03	UC
5	2		CLAYEY SAND WITH GRAVEL (SC), dark reddish brown, dense, slightly moist			44					9	131.2	3260.2		UU
10	3		No Recovery			87									
	4		POORLY GRADED GRAVEL WITH CLAY AND SAND (GP-GC), olive brown, dense, moist			41				11					
15	5		POORLY GRADED GRAVEL WITH CLAY AND SAND (GP-GC), brown and gray, medium dense, moist			18									
20	6		POORLY GRADED GRAVEL WITH CLAY AND SAND (GP-GC), brown, medium dense, wet			28									
	7														
25															

LOG - SHEAR AND UNCONF STRENGTH 12175.GPJ ENGEO INC.GDT 7/17/15

LOG OF BORING 1-B3

ENGEO INCORPORATED

Geotechnical Exploration
401 Alberto Way
Los Gatos, California
12175.000.000

DATE DRILLED: 6/27/2015
HOLE DEPTH: Approx. 34¾ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (): Approx. 338½ ft.

LOGGED / REVIEWED BY: I. McCreery / PCG
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	8		CLAYEY GRAVEL WITH SAND (GC), blue gray, medium dense, wet			27				14					
						157									
30	9		SHALE, very dark brown, weak (R2), very closely fractured												
	10														
			SHALE, very dark brown, weak (R2), very closely fractured			50 for 3 in.									
			Total depth approximately 34.75 feet bgs. Depth to groundwater not measured due to caving when removing augers. Backfilled with grout.												

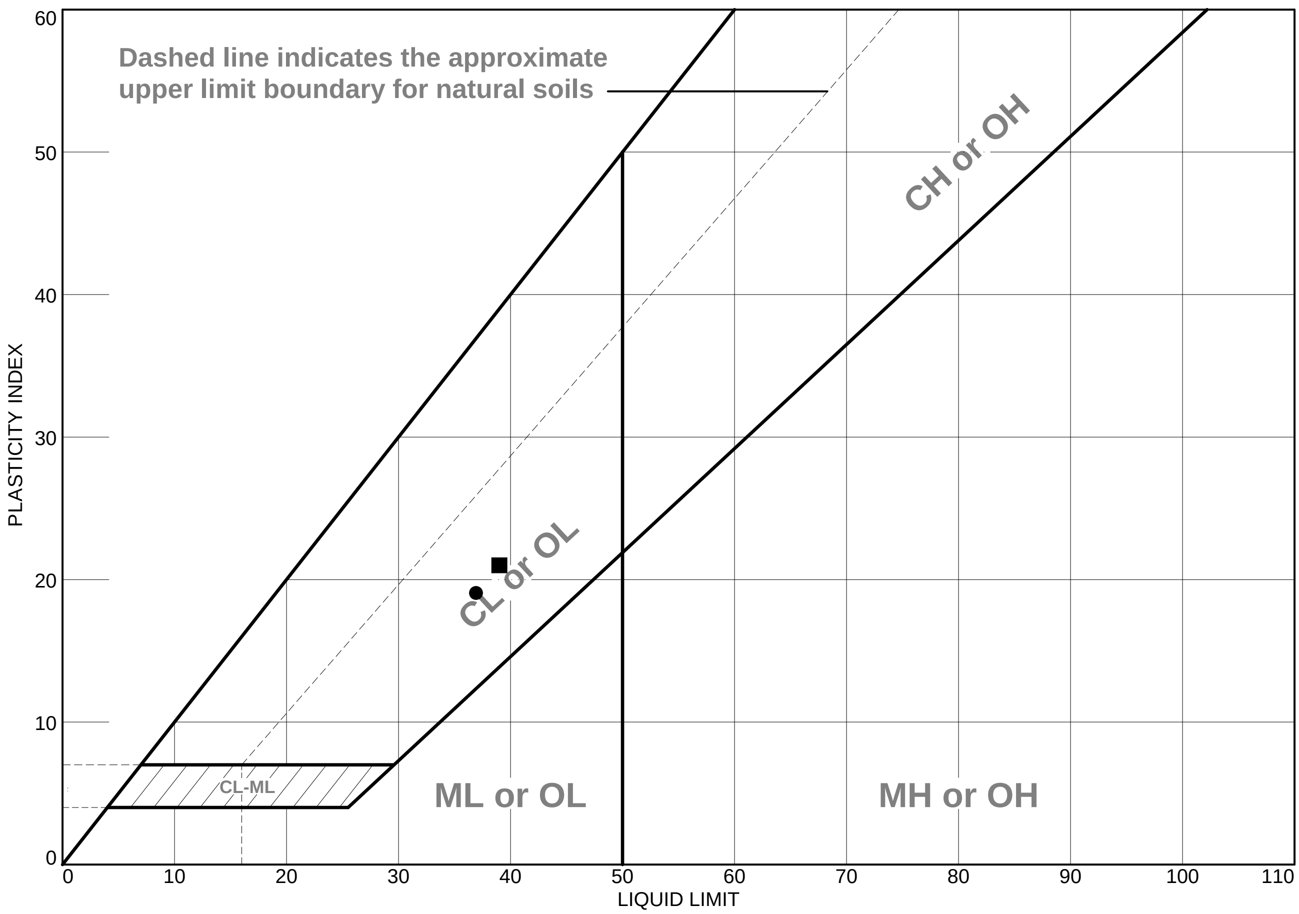
LOG - SHEAR AND UNCONF STRENGTH 12175.GPJ ENGEO INC.GDT 7/17/15

APPENDIX B

Laboratory Test Results
(ENGEO, 2015)

APPENDIX B

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	37	18	19			
■	See exploration logs	39	18	21			

Project No. 12175.000.000 **Client:** LP Aquisitions, LLC

Project: 401 Alberto Way, Los Gatos, Feasibility Study

●**Depth:** 4.5-5.0 feet **Sample Number:** 1-B1 @ 4.5-5

■**Depth:** 15.0-16.25 feet **Sample Number:** 1-B2 @ 15-16.25

Remarks:

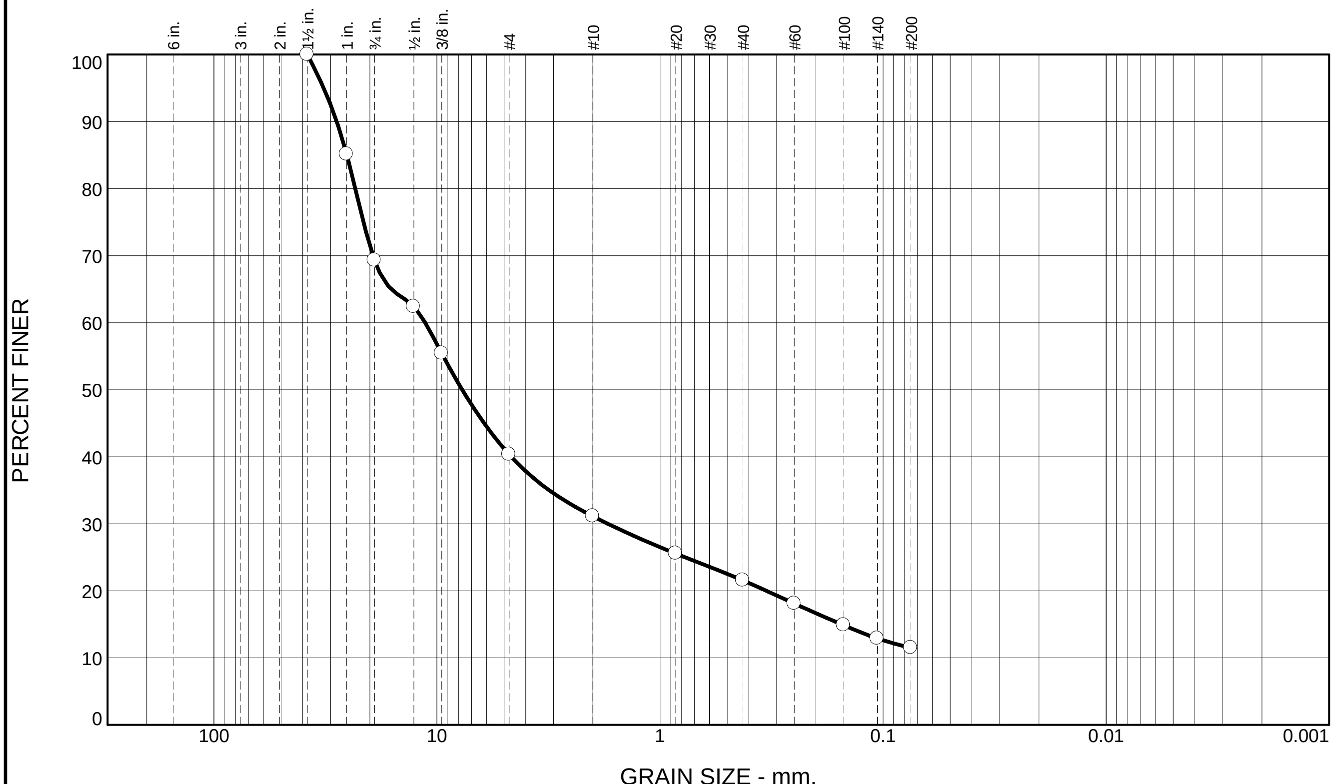
●ASTM D4318, Wet method

■ASTM D4318, Wet method

ENGEO INCORPORATED

Tested By: M. Liu **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	30.7	29.0	9.2	9.6	10.0	11.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2	100.0		
1	85.1		
3/4	69.3		
1/2	62.4		
3/8	55.4		
#4	40.3		
#10	31.1		
#20	25.6		
#40	21.5		
#60	18.1		
#100	14.9		
#140	12.9		
#200	11.5		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 28.1404 D_{85}= 25.3460 D_{60}= 11.2983
D_{50}= 7.7108 D_{30}= 1.7132 D_{15}= 0.1536
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913

Sample Number: 1-B2 @ 21-21.5 **Depth:** 21.0-21.5 feet

Date: 07/07/15

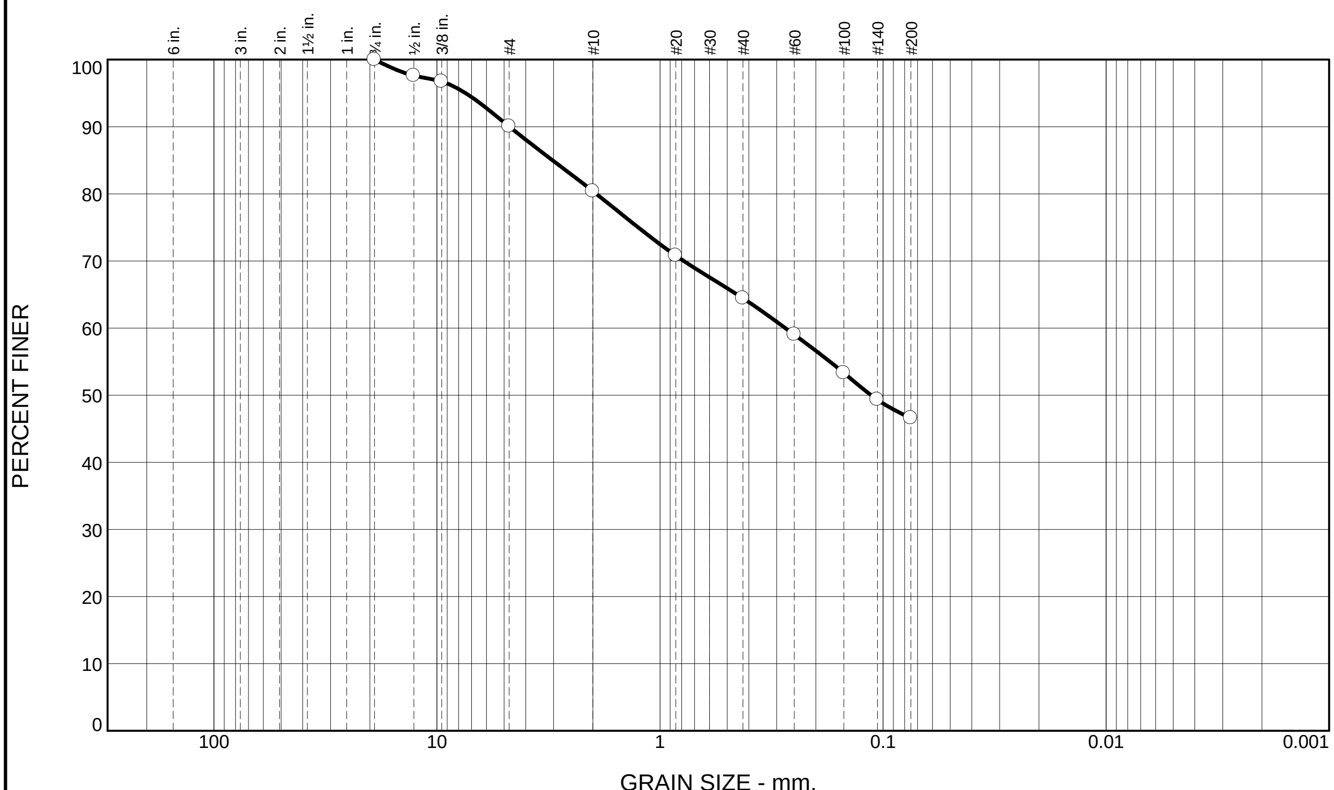
ENGEO INCORPORATED

Client: LP Aquisitions, LLC
Project: 401 Alberto Way, Los Gatos, Feasibility Study

Project No: 12175.000.000

Tested By: J. Lawton **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	10.0	9.6	16.0	17.8	46.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	97.6		
3/8	96.7		
#4	90.0		
#10	80.4		
#20	70.8		
#40	64.4		
#60	59.0		
#100	53.3		
#140	49.3		
#200	46.6		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 4.7364 D_{85}= 3.0312 D_{60}= 0.2740
D_{50}= 0.1132 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913

Sample Number: 1-B3 @ 3-3.5 **Depth:** 3.0-3.5 feet

Date: 07/07/15

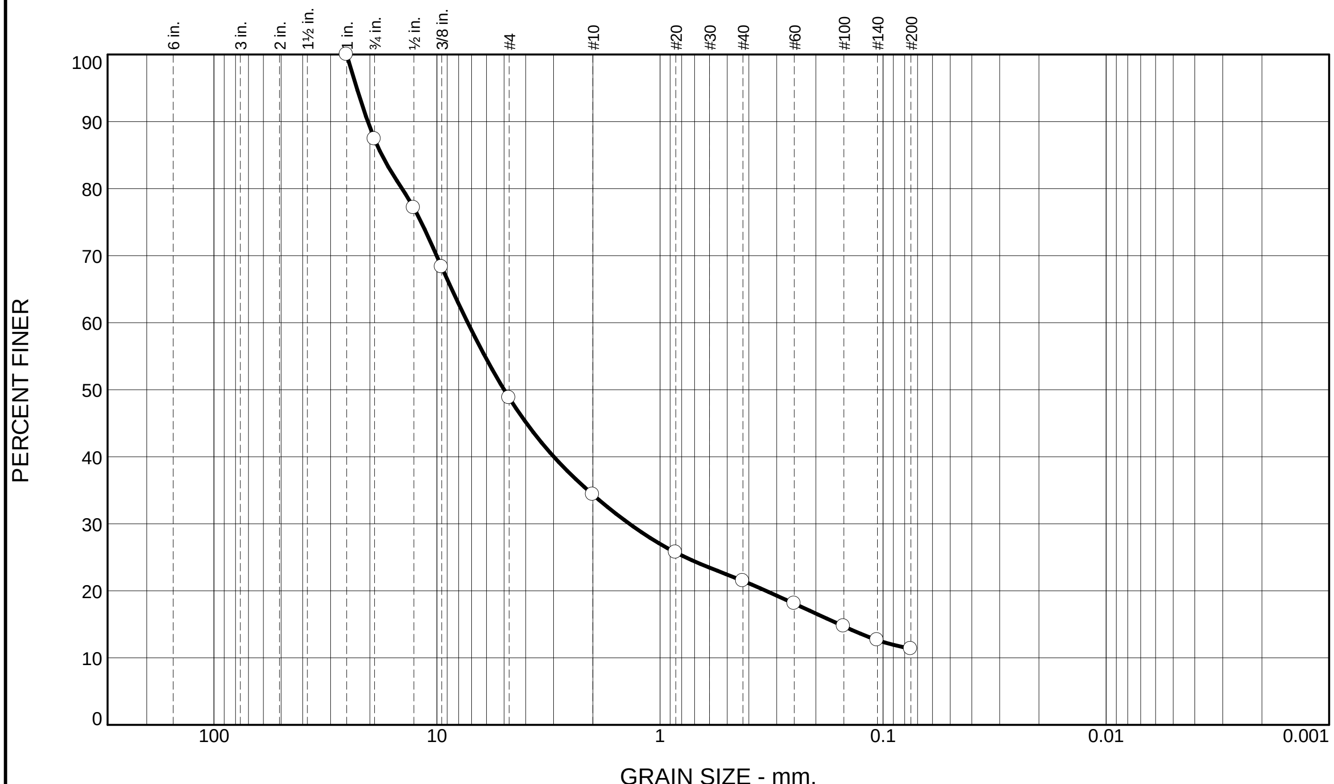
ENGEO INCORPORATED

Client: LP Aquisitions, LLC
Project: 401 Alberto Way, Los Gatos, Feasibility Study

Project No: 12175.000.000

Tested By: J. Lawton **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	12.6	38.6	14.4	12.9	10.2	11.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	87.4		
1/2	77.1		
3/8	68.3		
#4	48.8		
#10	34.4		
#20	25.7		
#40	21.5		
#60	18.1		
#100	14.7		
#140	12.6		
#200	11.3		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 20.4245 D_{85}= 17.6653 D_{60}= 7.2844
D_{50}= 5.0049 D_{30}= 1.3714 D_{15}= 0.1570
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913

Sample Number: 1-B3 @ 11.5-13 **Depth:** 11.5-13.0 feet

Date: 07/07/15

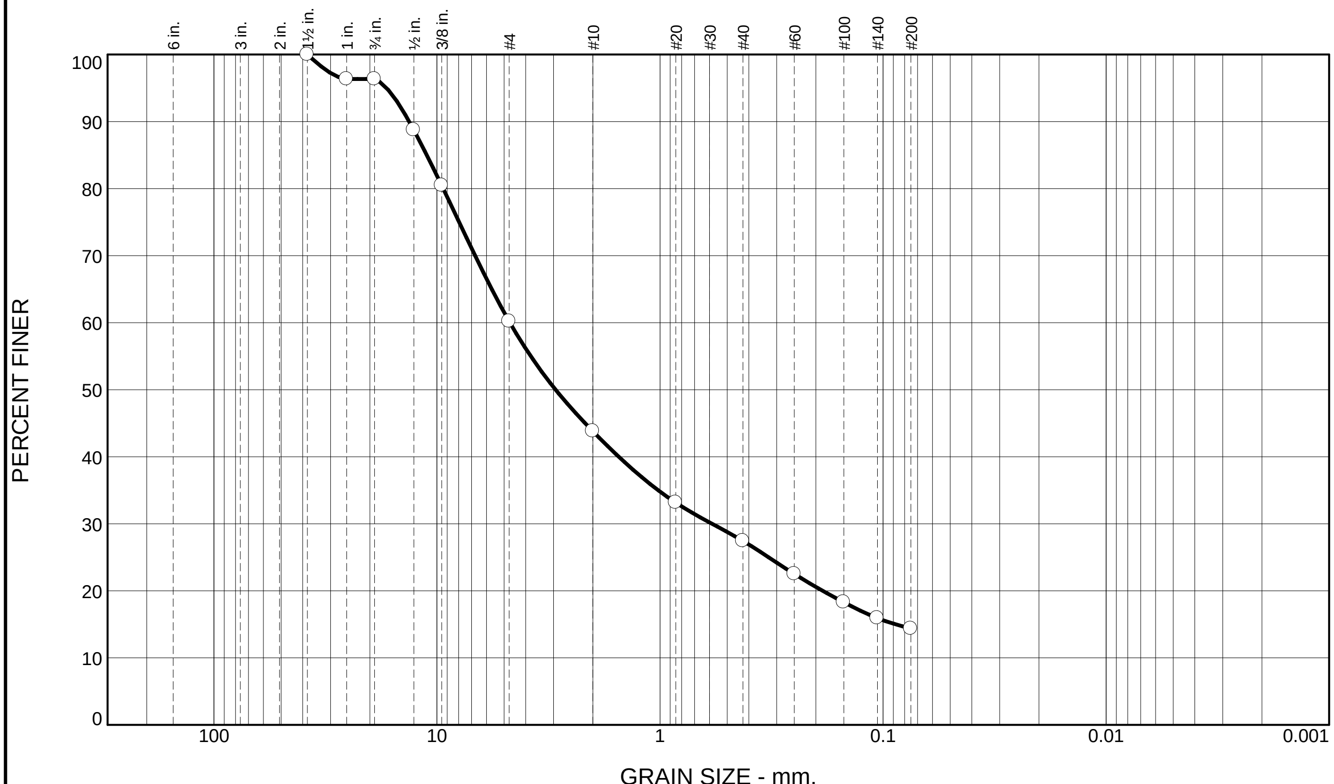
ENGEO INCORPORATED

Client: LP Aquisitions, LLC
Project: 401 Alberto Way, Los Gatos, Feasibility Study

Project No: 12175.000.000

Tested By: J. Lawton **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.7	36.1	16.4	16.4	13.0	14.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1-1/2	100.0		
1	96.3		
3/4	96.3		
1/2	88.7		
3/8	80.5		
#4	60.2		
#10	43.8		
#20	33.2		
#40	27.4		
#60	22.5		
#100	18.3		
#140	15.9		
#200	14.4		

* (no specification provided)

Material Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 13.3276 D_{85}= 11.1129 D_{60}= 4.7148
D_{50}= 2.9374 D_{30}= 0.5847 D_{15}= 0.0877
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913

Sample Number: 1-B3 @ 25.5-26.5 **Depth:** 25.5-26.5 feet

Date: 07/07/15

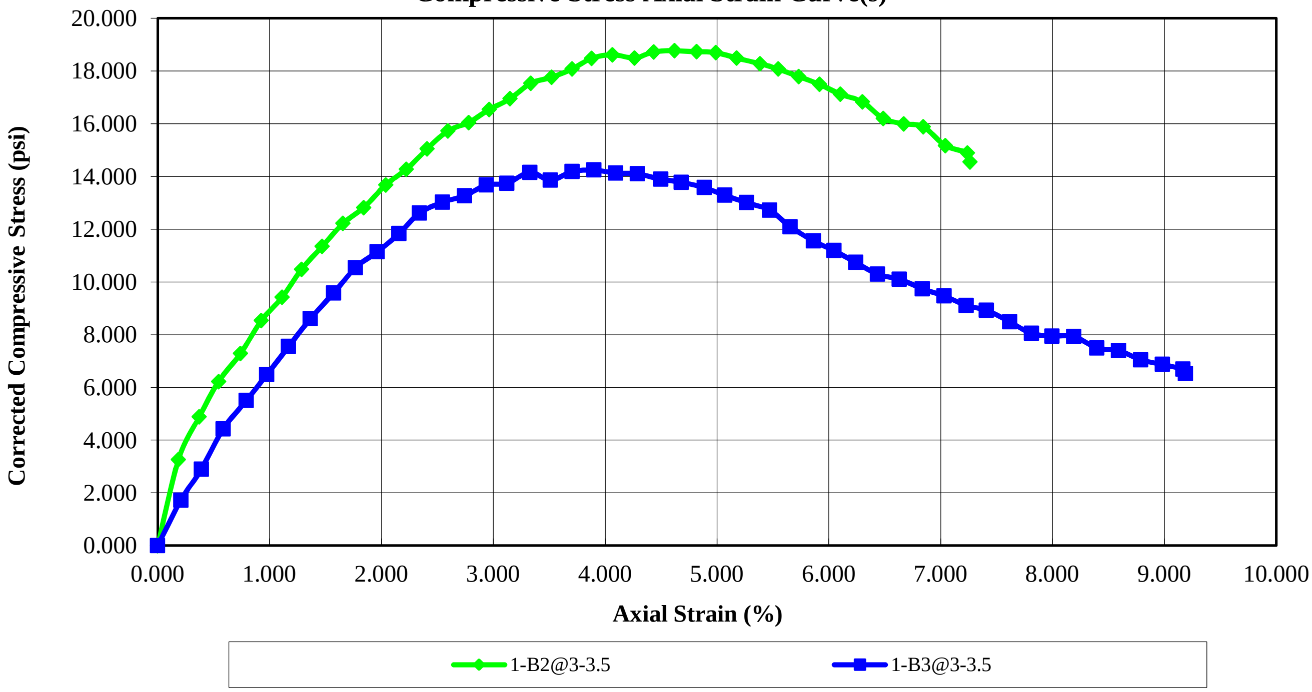
ENGEO INCORPORATED

Client: LP Aquisitions, LLC
Project: 401 Alberto Way, Los Gatos, Feasibility Study

Project No: 12175.000.000

Tested By: J. Lawton **Checked By:** G. Criste

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B2@3-3.5	1-B3@3-3.5
Moisture Content (%)	9.9	12.6
Dry Density (pcf)	102.4	102.5
Saturation (%)	42.68	54.25
Void Ratio	0.62	0.61
Diameter (in)	2.397	2.394
Height (in)	5.653	5.372
Height-To-Diameter Ratio	2.358	2.244
TEST DATA		
Unconfined Compressive Strength (psf)	2702.838	2050.721
Undrained Shear Strength (psf)	1351.419	1025.360
Strain Rate (in./min.)	0.05	0.05
Specific Gravity	2.650	2.650
Strain at Failure (%)	4.62	3.90

Test Remarks

SPECIMEN	DESCRIPTION
1-B2@3-3.5	See exploration logs
1-B3@3-3.5	See exploration logs

PROJECT NAME: 401 Alberto Way, Los Gatos, Feasibility Study
PROJECT NO: 12175.000.000
CLIENT: LP Aquisitions, LLC
LOCATION: Los Gatos, CA
PHASE NO: 002

Test Date: 07/06/15
Tested By: G. Criste
Reviewed By: D. Seibold

ENGEO Expect Excellence

EN **GEO**

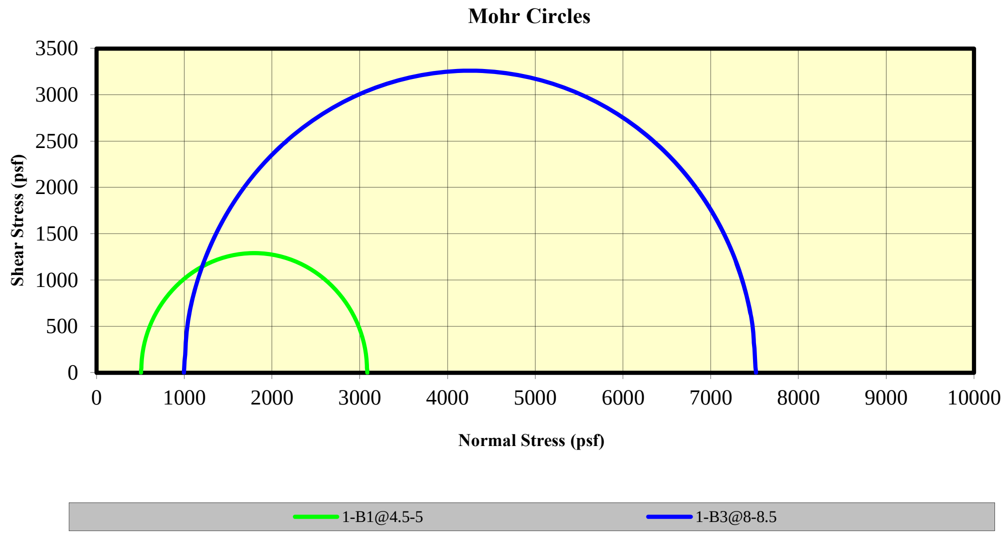
Unconsolidated Undrained Triaxial Test (ASTM D2850)

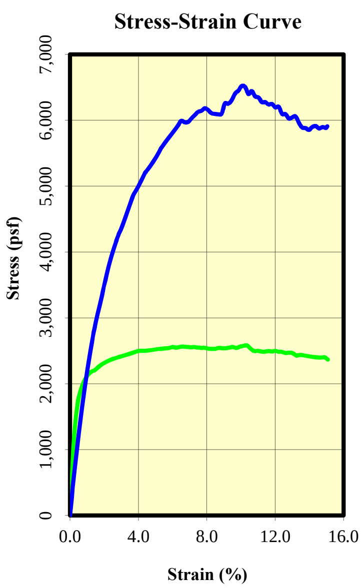
Date: 07/06/15

Checked By: D. Seibold

Date: 07/06/15

Tested By: G. Criste





Before Test	Specimen 1-B1@4.5-5	1-B3@8-8.5		
Water Content (%)	9.38	9.04		
Dry Density (pcf)	98.69	131.24		
Saturation (%)	36.75	80.68		
Void Ratio	0.68	0.31		
Diameter (in)	2.380	2.404		
Height (in)	5.045	5.302		
Liquid Limit	-	-		
Plastic Limit	-	-		
Specific Gravity	2.650	2.750		
Height-to-Diameter Ratio	2.120	2.205		
After Test	**1-B1@4.5-5**	**1-B3@8-8.5**		
Water Content (%)	9.38	9.04		
Saturation (%)	36.75	80.68		
Strain Rate (in/min)	0.05	0.05		
Peak Deviator Stress (psf)	2581.7	6520.3		
Axial Strain @ Failure (%)	10.343	10.038		
	Cell Pressure			
Cell (psf)	504.0	993.6		
Back (psf)	n/a	n/a		
	Principle Stresses at Failure			
σ1 (psf)	3085.7	7513.9		
σ3 (psf)	504.0	993.6		

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	0.0		1290.8	3260.2		
Friction Angle Ø	0.00		n/a	n/a		

Project Information			
Project Name:	401 Alberto Way, Los Gatos, Feasibility Study		
Project Number:	12175.000.000	Job Number:	12175.000.000
Location:	Los Gatos, CA	Boring Number:	Multiple
Client:	LP Aquisitions, LLC	Sample Number:	Multiple
Description:	See exploration logs		

APPENDIX C

Corrosivity Test Results
(CERCO, 2015)

APPENDIX C

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: ENGEO Incorporated
Client's Project No.: 12175.000.000
Client's Project Name: Alberto Way, Los Gatos
Date Sampled: 27-Jun-15
Date Received: 2-Jul-15
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 13-Jul-2015

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1507016-001	1-B1 @ 8.5'-10'	320	7.46	-	5,300	-	N.D.	28
1507016-002	1-B3 @ 20'-21.5'	380	7.47	-	5,900	-	N.D.	32

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	10-Jul-2015	9-Jul-2015	-	8-Jul-2015	-	10-Jul-2015	10-Jul-2015

Cheryl McMillen

Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis
N.D. - None Detected

Quality Control Summary - **All laboratory quality control parameters were found to be within established limits**

APPENDIX D

Liquefaction Analysis

APPENDIX D

401 Alberto Way, Los Gatos

Liquefaction Evaluation - Youd 2001, Seed 2003, I&B 2008 Methods -

Note, if sloping ground and non-zero statis shear stress exist, user may chose to change value of kalpha

Input

Yellow cells are calculated

Green cells require user input - reference respective papers for details
Corrdction factors on "Driving Force" and "Resisting Force" sheets require user input

Water Table depth at time of Exploration	Water Table depth at time of Liquefaction	amax/g	Mw	$V_{s*40'}$
21	15	1.00	8	1180

* V_{S40} = Avg shear wave velocity in upper 40 feet expressed in ft/s

					At time of Exploration		At time of Liquefaction	
Boring Designation	Depth [ft]	Soil Type	N_m [Blows/ft]	FC	Total Stress [psf]	Effective Stress [psf]	Total Stress [psf]	Effective Stress [psf]
1-B2	15	SC	44	20	1875	1875	1875	1875
1-B2	20	GP-GC	98	12	2500	2500	2500	2188
1-B2	25	GP-GC	54	12	3125	2875.4	3125	2501
1-B2	30	GP-GC	61	12	3750	3188.4	3750	2814
1-B3	15	GP-GC	18	12	1875	1875	1875	1875
1-B3	20	GP-GC	28	12	2500	2500	2500	2188
1-B3	25	GP-GC	27	12	3125	2875.4	3125	2501
						0	0	0
						0	0	0
						0	0	0
						0		0
						0		0
						0		0
						0		0

N_m = Measured SPT Blow Count

YOUD 2001 Methodology Results

Boring Designation	Depth	CRR	CSR	FS
1-B2	15	TDL	0.63	TDL
1-B2	20	TDL	0.71	TDL
1-B2	25	TDL	0.76	TDL
1-B2	30	TDL	0.80	TDL
1-B3	15	0.25	0.63	0.40
1-B3	20	TDL	0.71	TDL
1-B3	25	TDL	0.76	TDL
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!

TDL = Too Dense to Liquefy based on blowcount criteria

ENGEO Expect Excellence 40 years

401 Alberto Way, Los Gatos

SEED 2003 Methodology Results

Boring Designation	Depth	CRR	CSR			Calculated FS		
			mean rd	rd + sigma	rd - sigma	mean rd	rd + sigma	rd - sigma
1-B2	15	1.81	0.70	0.75	0.65	FS>2.5	2.43	FS>2.5
1-B2	20	THC	0.84	0.92	0.76	THC	THC	THC
1-B2	25	1.90	0.95	1.05	0.84	2.00	1.80	2.26
1-B2	30	2.88	1.03	1.17	0.90	FS>2.5	2.48	FS>2.5
1-B3	15	0.15	0.69	0.74	0.64	0.22	0.21	0.24
1-B3	20	0.27	0.85	0.93	0.77	0.32	0.30	0.36
1-B3	25	0.23	0.96	1.07	0.86	0.24	0.22	0.27
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

THC = CRR capped at 4, in high seismicity cases, verify

Idriss & Boulanger 2008 Methodology Results

Boring Designation	Depth	CRR	CSR	FS
1-B2	15	THC	-1.53	THC
1-B2	20	THC	0.82	THC
1-B2	25	THC	0.80	THC
1-B2	30	THC	0.84	THC
1-B3	15	0.29	0.71	0.41
1-B3	20	0.93	0.83	1.13
1-B3	25	0.58	0.91	0.64
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!

THC = CRR capped at 4, in high seismicity cases, verify

ENGEO Expect Excellence 40 years

Liquefaction Evaluation - Driving Force

Boring No.	0

Youd 2001

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	rd	CSR
1-B2	15	1875	1875	0.968	0.63
1-B2	20	2500	2188	0.956	0.71
1-B2	25	3125	2501	0.941	0.76
1-B2	30	3750	2814	0.919	0.80
1-B3	15	1875	1875	0.968	0.63
1-B3	20	2500	2188	0.956	0.71
1-B3	25	3125	2501	0.941	0.76
0	0	0	0	1.000	#DIV/0!
0	0	0	0	1.000	#DIV/0!
0	0	0	0	1.000	#DIV/0!
0	0	0	0	1.000	#DIV/0!
0	0	0	0	1.000	#DIV/0!
0	0	0	0	1.000	#DIV/0!

SEED 2003

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	CN	(N1)60	Ce	Cb	Cr	Cs
1-B2	15	1875	1875	1.03	50	1.1	1.15	0.87	1
1-B2	20	2500	2500	0.89	100	1.1	1.15	0.9	1
1-B2	25	3125	2875	0.83	54	1.1	1.15	0.94	1
1-B2	30	3750	3188	0.79	59	1.1	1.15	0.97	1
1-B3	15	1875	1875	1.03	20	1.1	1.15	0.87	1
1-B3	20	2500	2500	0.89	29	1.1	1.15	0.9	1
1-B3	25	3125	2875	0.83	27	1.1	1.15	0.94	1
0	0	0	0	#DIV/0!	#DIV/0!				
0	0	0	0	#DIV/0!	#DIV/0!				
0	0	0	0	#DIV/0!	#DIV/0!				
0	0	0	0	#DIV/0!	#DIV/0!				
0	0	0	0	#DIV/0!	#DIV/0!				
0	0	0	0	#DIV/0!	#DIV/0!				

DWF
0.91

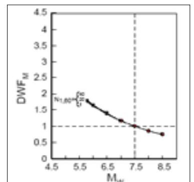
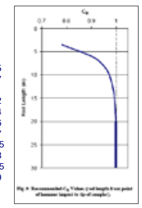


Fig. 13: Recommended Magnitude-Correlated Duration Weighting Factor as a Function of $N_{1,60}$

Rod Length (feet)	CR
9	0.8
12	0.85
15	0.87
18	0.9
21	0.92
24	0.94
27	0.96
30	0.97
33	0.975
36	0.98
39	0.985
45	0.99
60	1
100	1



Boring Designation	Depth	Cfines	(N1)60cs	rd	sigma	rd +sigma	rd -sigma	f	K sigma	K alpha ~
1-B2	15	1.10	55	1.000	0.072	1.072	0.928	0.80	1.02	1.00
1-B2	20	1.05	105	1.000	0.092	1.092	0.908	0.80	0.97	1.00
1-B2	25	1.06	57	1.000	0.111	1.111	0.889	0.80	0.94	1.00
1-B2	30	1.06	63	1.000	0.130	1.130	0.871	0.80	0.92	1.00
1-B3	15	1.08	22	1.000	0.072	1.072	0.928	0.71	1.04	1.00
1-B3	20	1.07	30	1.000	0.092	1.092	0.908	0.75	0.96	1.00
1-B3	25	1.07	29	1.000	0.111	1.111	0.889	0.74	0.92	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00

Boring Designation	mean value of rd				rd + sigma				rd - sigma			
	CSReq	CSRN	CSR*	CSR*α>0	CSReq	CSRN	CSR*	CSR*α>0	CSReq	CSRN	CSR*	CSR*α>0
1-B2	0.65	0.71	0.70	0.70	0.70	0.76	0.75	0.75	0.60	0.66	0.65	0.65
1-B2	0.74	0.81	0.84	0.84	0.81	0.89	0.92	0.92	0.67	0.74	0.76	0.76
1-B2	0.81	0.89	0.95	0.95	0.90	0.99	1.05	1.05	0.72	0.79	0.84	0.84
1-B2	0.87	0.95	1.03	1.03	0.98	1.07	1.17	1.17	0.75	0.83	0.90	0.90
1-B3	0.65	0.71	0.69	0.69	0.70	0.76	0.74	0.74	0.60	0.66	0.64	0.64
1-B3	0.74	0.81	0.85	0.85	0.81	0.89	0.93	0.93	0.67	0.74	0.77	0.77
1-B3	0.81	0.89	0.96	0.96	0.90	0.99	1.07	1.07	0.72	0.79	0.86	0.86
0.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

~ K alpha = 1.0 for level ground conditions only (no static shear stress)

I&B 2008

MSF
0.88

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	rd	CSR	Csigma	K sigma	K alpha ~	CSR7.5
1-B2	15	1875	1875	0.978	0.64	-12.185	-0.47	1.00	-1.53
1-B2	20	2500	2188	0.966	0.72	-0.119	1.00	1.00	0.82
1-B2	25	3125	2501	0.953	0.77	-1.022	1.10	1.00	0.80
1-B2	30	3750	2814	0.938	0.81	-0.403	1.10	1.00	0.84
1-B3	15	1875	1875	0.978	0.64	0.149	1.02	1.00	0.71
1-B3	20	2500	2188	0.966	0.72	0.224	0.99	1.00	0.83
1-B3	25	3125	2501	0.953	0.77	0.196	0.97	1.00	0.91
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!

~ K alpha = 1.0 for level ground conditions only (no static shear stress)

Liquefaction Evaluation - Resisting Force

Boring No.	0

YOUD 2001

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	CN	Ce	Cb	Cr	Cs	(N1)60
1-B2	15	1875	1875	1.03	1.1333333	1.15	0.95	1	56
1-B2	20	2500	2500	0.89	1.1333333	1.15	0.95	1	109
1-B2	25	3125	2875	0.83	1.1333333	1.15	0.95	1	56
1-B2	30	3750	3188	0.79	1.1333333	1.15	1	1	63
1-B3	15	1875	1875	1.03	1.1333333	1.15	0.95	1	23
1-B3	20	2500	2500	0.89	1.1333333	1.15	0.95	1	31
1-B3	25	3125	2875	0.83	1.1333333	1.15	0.95	1	28
0	0	0	0	#DIV/0!					#DIV/0!
0	0	0	0	#DIV/0!					#DIV/0!
0	0	0	0	#DIV/0!					#DIV/0!
0	0	0	0	#DIV/0!					#DIV/0!
0	0	0	0	#DIV/0!					#DIV/0!
0	0	0	0	#DIV/0!					#DIV/0!

MSF
0.8474023

Boring Designation	Depth	alpha	beta	(N1)60cs	f	K sigma	K alpha ~	CRR-B	CRR-M
1-B2	15	3.61	1.08	64	0.80	1.02	1.00	0.44	TDL
1-B2	20	1.55	1.03	114	0.80	0.97	1.00	0.82	TDL
1-B2	25	1.55	1.03	59	0.80	0.94	1.00	0.39	TDL
1-B2	30	1.55	1.03	67	0.80	0.92	1.00	0.46	TDL
1-B3	15	1.55	1.03	25	0.73	1.03	1.00	0.30	0.25
1-B3	20	1.55	1.03	34	0.77	0.96	1.00	2.42	TDL
1-B3	25	1.55	1.03	30	0.75	0.93	1.00	0.49	TDL
0.00	0	0.00	1.00	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!	#DIV/0!
0.00	0	0.00	1.00	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!	#DIV/0!
0.00	0	0.00	1.00	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!	#DIV/0!
0.00	0	0.00	1.00	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!	#DIV/0!
0.00	0	0.00	1.00	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!	#DIV/0!
0.00	0	0.00	1.00	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!	#DIV/0!

~ K alpha = 1.0 for level ground conditions only (no static shear stress)

SEED 2003

Boring Designation	Depth	CRR
1-B2	15	1.81
1-B2	20	74.93
1-B2	25	1.90
1-B2	30	2.88
1-B3	15	0.15
1-B3	20	0.27
1-B3	25	0.23
0	0	#DIV/0!
0	0	#DIV/0!
0	0	#DIV/0!
0	0	#DIV/0!
0	0	#DIV/0!
0	0	#DIV/0!

I&B 2008

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	(N)60	Ce	Cb	Cr	Cs
1-B2	15	1875	1875	54	1.1333333	1.15	0.95	1
1-B2	20	2500	2500	121	1.1333333	1.15	0.95	1
1-B2	25	3125	2875	67	1.1333333	1.15	0.95	1
1-B2	30	3750	3188	80	1.1333333	1.15	1	1
1-B3	15	1875	1875	22	1.1333333	1.15	0.95	1
1-B3	20	2500	2500	35	1.1333333	1.15	0.95	1
1-B3	25	3125	2875	33	1.1333333	1.15	0.95	1
0	0	0	0	0				
0	0	0	0	0				
0	0	0	0	0				
0	0	0	0	0				
0	0	0	0	0				
0	0	0	0	0				

Iterations of CN Value

Boring Designation	Depth	CN i	N1-1	Cn1	N1-2	Cn2	N1-3	Cn3	N14	CN	(N1)60
1-B2	15	1.01	46	1.02	46	1.02	46	1.02	46	1.02	55
1-B2	20	1.01	46	0.94	46	0.94	46	0.94	46	0.94	114
1-B2	25	0.94	46	0.91	46	0.91	46	0.91	46	0.91	61
1-B2	30	0.95	46	0.88	46	0.88	46	0.88	46	0.88	70
1-B3	15	1.03	22.901497	1.03	22.894161	1.03	22.894248	1.03	22.894247	1.03	23
1-B3	20	0.93	32.194547	0.93	32.0767	0.93	32.070987	0.93	32.07071	0.93	32
1-B3	25	0.88	29.54906	0.88	29.265362	0.87	29.244026	0.87	29.242418	0.87	29
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

Boring Designation	Depth	Delta N	(N1)60cs	CRR-B
1-B2	15	4.48	60	#########
1-B2	20	2.07	116	#########
1-B2	25	2.07	63	#########
1-B2	30	2.07	72	#########
1-B3	15	2.07	25	0.29
1-B3	20	2.07	34	0.93
1-B3	25	2.07	31	0.58
0.00	0	0.00	#DIV/0!	#DIV/0!
0.00	0	0.00	#DIV/0!	#DIV/0!
0.00	0	0.00	#DIV/0!	#DIV/0!
0.00	0	0.00	#DIV/0!	#DIV/0!
0.00	0	0.00	#DIV/0!	#DIV/0!
0.00	0	0.00	#DIV/0!	#DIV/0!

APPENDIX E

Supplemental Recommendations

APPENDIX E

SUPPLEMENTAL RECOMMENDATIONS

TABLE OF CONTENTS

GENERAL INFORMATION

PREFACE

These supplemental recommendations are intended as a guide for earthwork and are in addition to any previous earthwork recommendations made by the Geotechnical Engineer. If there is a conflict between these supplemental recommendations and any previous recommendations, it should be immediately brought to the attention of ENGEO. Testing standards identified in this document shall be the most current revision (unless stated otherwise).

DEFINITIONS

Backfill	Soil, rock or soil-rock material used to fill excavations and trenches.
Drawings	Documents approved for construction which describe the work.
The Geotechnical Engineer	The project geotechnical engineering consulting firm, its employees, or its designated representatives.
Engineered Fill	Fill upon which the Geotechnical Engineer has made sufficient observations and tests to confirm that the fill has been placed and compacted in accordance with geotechnical engineering recommendations.
Fill	Soil, rock, or soil-rock materials placed to raise the grades of the site or to backfill excavations.
Imported Material	Soil and/or rock material which is brought to the site from offsite areas.
Onsite Material	Soil and/or rock material which is obtained from the site.
Optimum Moisture	Water content, percentage by dry weight, corresponding to the maximum dry density as determined by ASTM D-1557.
Relative Compaction	The ratio, expressed as a percentage, of the in-place dry density of the fill or backfill material as compacted in the field to the maximum dry density of the same material as determined by ASTM D-1557.
Select Material	Onsite and/or imported material which is approved by the Geotechnical Engineer as a specific-purpose fill.

PART I - EARTHWORK

1.1 GENERAL

1.1.1 WORK COVERED

Supplemental recommendations for performing earthwork and grading. Activities include:

- ✓ Site Preparation and Demolition
- ✓ Excavation
- ✓ Grading
- ✓ Backfill of Excavations and Trenches
- ✓ Engineered Fill Placement, Moisture Conditioning, and Compaction

1.1.2 CODES AND STANDARDS

The contractor should perform their work complying with applicable occupational safety and health standards, rules, regulations, and orders. The Occupational Safety and Health Standards (OSHA) Board is the only agency authorized in the State to adopt and enforce occupational safety and health standards (Labor Code § 142 et seq.). The owner, their representative and contractor are responsible for site safety; ENGEO representatives are not responsible for site safety.

Excavating, trenching, filling, backfilling, shoring and grading work should meet the minimum requirements of the applicable Building Code, and the standards and ordinances of state and local governing authorities.

1.1.3 TESTING AND OBSERVATION

Site preparation, cutting and shaping, excavating, filling, and backfilling should be carried out under the testing and observation of ENGEO. ENGEO shall be retained to perform appropriate field and laboratory tests to check compliance with the recommendations. Any fill or backfill that does not meet the supplemental recommendations shall be removed and/or reworked, until the supplemental recommendations are satisfied.

Tests for compaction shall be made in accordance with test procedures outlined in ASTM D-1557, as applicable, unless other testing methods are deemed appropriate by ENGEO. These and other tests shall be performed in accordance with accepted testing procedures, subject to the engineering discretion of ENGEO.

1.2 MATERIALS

1.2.1 STANDARD

Materials, tools, equipment, facilities, and services as required for performing the required excavating, trenching, filling and backfilling should be furnished by the Contractor.

1.2.2 ENGINEERED FILL AND BACKFILL

Material to be used for engineered fill and backfill should be free from organic matter and other deleterious substances, and of such quality that it will compact thoroughly without excessive voids when watered and rolled.

Unless specified elsewhere by ENGEO, engineered fill and backfill shall be free of significant organics, or any other unsatisfactory material. In addition, engineered fill and backfill shall comply with the grading requirements shown in the following table:

TABLE 1.2.2-1
Engineered Fill and Backfill Requirements

US Standard Sieve	Percentage Passing
3"	100
No. 4	35–100
No. 30	20–100

Earth materials to be used as engineered fill and backfill shall be cleared of debris, rubble and deleterious matter. Rocks and aggregate exceeding the maximum allowable size shall be removed from the site. Rocks of maximum dimension in excess of two-thirds of the lift thickness shall be removed from any fill material to the satisfaction of ENGEO.

ENGEO shall be immediately notified if potential hazardous materials or suspect soils exhibiting staining or odor are encountered. Work activities shall be discontinued within the area of potentially hazardous materials. ENGEO shall be notified at least 72 hours prior to the start of filling and backfilling operations. Materials to be used for filling and backfilling shall be submitted to ENGEO no less than 10 days prior to intended delivery to the site. Unless specified elsewhere by ENGEO, where conditions require the importation of low expansive fill material, the material shall be an inert, low to non-expansive soil, or soil-rock material, free of organic matter and meeting the following requirements:

TABLE 1.2.2-2
Imported Fill Material Requirements

GRADATION (ASTM D-421)	SIEVE SIZE	PERCENT PASSING
	2-inch	100
	#200	15 - 70
PLASTICITY (ASTM D-4318)	Plasticity Index < 12	
ORGANIC CONTENT (ASTM D-2974)	Less than 2 percent	

A sample of the proposed import material should be submitted to ENGEO no less than 10 days prior to intended delivery to the site.

1.2.3 SUBDRAINS

A subdrain system is an underground network of piping used to remove water from areas that collect or retain surface water or subsurface water. Subsurface water is collected by allowing water into the pipe through perforations. Subdrain systems may drain and discharge to an appropriate outlet such as storm drain, natural swales or drainage, etc.. Details for subdrain systems may vary depending on many items, including but not limited to site conditions, soil types, subdrain spacing, depth of the pipe and pervious medium, as well as pipe diameter.

1.2.3A Pipe

Subdrain pipe shall conform with these supplemental recommendations unless specified elsewhere by ENGEO. Perforated pipe for various depths shall be manufactured in accordance with the following requirements:

TABLE 1.2.3A-1
Perforated Pipe Requirements

Pipe Type	Standard	Typical Sizes (inches)	Pipe Stiffness (psi)
Pipe Stiffness above 200 psi (Below 50 feet of Finished Grade)			
ABS SDR 15.3		4 to 6	450
PVC Schedule 80	ASTM D1785	3 to 10	530
Pipe Stiffness between 100 psi and 150 psi (Between 15 and 50 feet of Finished Grade)			
ABS SDR 23.5	ASTM D2751	4 to 6	150
PVC SDR 23.5	ASTM D3034	4 to 6	153
PVC Schedule 40	ASTM D1785	3 to 10	135
ABS Schedule 40/DWV	ASTM D1527 & D2661	3 to 10	
Pipe Stiffness between 45 psi and 50 psi* (Between 0 to 15 feet of Finished Grade)			
PVC A-2000	ASTM F949	4 to 10	50
PVC SDR 35	ASTM D3034	4 to 8	46
ABS SDR 35	ASTM D2751	4 to 8	45
Corrugated PE	AASHTO M294 Type S	4 to 10	45

*Pipe with a stiffness less than 45 psi should not be used.

Other pipes not listed in the table above shall be submitted for review by the Geotechnical Engineer not less 72 hours before proposed use.

1.2.3B Outlets and Risers

Subdrain outlets and risers must be fabricated from the same material as the subdrain pipe. Outlet and riser pipe and fittings must not be perforated. Covers must be fitted and bolted into the riser pipe or elbow. Covers must seat uniformly and not be subject to rocking.

1.2.3C Permeable Material

Permeable material shall generally conform to Caltrans Standard Specification unless specified otherwise by ENGEO. Class 2 permeable material shall comply with the gradation requirements shown in the following table.

TABLE 1.2.3C-1
Class 2 Permeable Material Grading Requirements

Sieve sizes	Percentage passing
1"	100
3/4"	90 to 100
3/8"	40 to 100
No. 4	25 to 40
No. 8	18 to 33
No. 30	5 to 15
No. 50	0 to 7
No. 200	0 to 3

1.2.3D Filter Fabric

Filter fabric shall meet the following Minimum Average Roll Values unless specified elsewhere by ENGEO.

Grab Strength (ASTM D-4632) .... 180 lbs
Mass per Unit Area (ASTM D-4751)..................................... 6 oz/yd^2
Apparent Opening Size (ASTM D-4751)........... 70-100 U.S. Std. Sieve
Flow Rate (ASTM D-4491)..80 gal/min/ft^2
Puncture Strength (ASTM D-4833) .... 80 lbs

Areas to receive filter fabric must comply with the compaction and elevation tolerance specified for the material involved. Handle and place filter fabric under the manufacturer's instructions. Align and place filter fabric without wrinkles.

Overlap adjacent roll ends of filter fabric in accordance with manufacturer’s recommendations. The preceding roll must overlap the following roll in the direction that the permeable material is being spread. Completely replace torn or punctured sections damaged during placement or repair by placing a piece of filter fabric that is large enough to cover the damaged area and comply with the overlap specified. Cover filter fabric with the thickness of overlying material shown within 72 hours of placing the fabric.

1.2.4 GEOCOMPOSITE DRAINAGE

Geocomposite drainage is a prefabricated material that includes filter fabric and plastic pipe. Filter fabric must be Class A. The drain shall be of composite construction consisting of a supporting structure or drainage core material surrounded by a geotextile. The geotextile shall encapsulate the drainage core and prevent random soil intrusion into the drainage structure. The drainage core material shall consist of a three-dimensional polymeric material with a structure that permits flow along the core laterally. The core structure shall also be constructed to permit flow regardless of the water inlet surface. The drainage core shall provide support to the geotextile.

A geotextile flap shall be provided along drainage core edges. This flap shall be of sufficient width for sealing the geotextile to the adjacent drainage structure edge to prevent soil intrusion into the structure during and after installation. The geotextile shall cover the full length of the core. The geocomposite core shall be furnished with an approved method of constructing and connecting with outlet pipes. If the fabric on the geocomposite drain is torn or punctured, replace the damaged section completely. The specific drainage composite material and supplier shall be preapproved by ENGEO.

The Contractor shall submit a manufacturer's certification that the geocomposite meets the design properties and respective index criteria measured in full accordance with applicable test methods. The manufacturer's certification shall include a submittal package of documented test results that confirm the design values. In case of dispute over validity of design values, the Contractor will supply design property test data from a laboratory approved by ENGEO, to support the certified values submitted.

Geocomposite material suppliers shall provide a qualified and experienced representative onsite to assist the Contractor and ENGEO at the start of construction with directions on the use of drainage composite. If there is more than one application on a project, this criterion will apply to construction of the initial application only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining applications. The soil surface against which the geocomposite is to be placed shall be free of debris and inordinate irregularities that will prevent intimate contact between the soil surface and the drain.

Edge seams shall be formed by utilizing the flap of the geotextile extending from the geocomposite's edge and lapping over the top of the fabric of the adjacent course. The fabric flap shall be securely fastened to the adjacent fabric by means of plastic tape or non-water-soluble construction adhesive, as recommended by the supplier. To prevent soil intrusion, exposed edges of the geocomposite drainage core edge must be covered.

Approved backfill shall be placed immediately over the geocomposite drain. Backfill operations should be performed to not damage the geotextile surface of the drain. Also during operations, avoid excessive settlement of the backfill material. The geocomposite drain, once installed, shall not be exposed for more than 7 days prior to backfilling.

PART II - GEOGRID SOIL REINFORCEMENT

Geogrid soil reinforcement (geogrid) shall be submitted to ENGEO and should be approved before use. The geogrid shall be a regular network of integrally connected polymer tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil or rock. The geogrid structure shall be dimensionally stable and able to retain its geometry under construction stresses and shall have high resistance to damage during construction to ultraviolet degradation and to chemical and biological degradation encountered in the soil being reinforced. The geogrids shall have an Allowable Tensile Strength (T_a) and Pullout Resistance, for the soil type(s) as specified on design plans.

The contractor shall submit a manufacturer's certification that the geogrids supplied meet plans and project specifications. The contractor shall check the geogrid upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the geogrid shall be protected from temperatures greater than 140°F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geogrid will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geogrid damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

Geogrid material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project, for a minimum of three days, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). Geogrid reinforcement may be joined with mechanical connections or overlaps as recommended and approved by the manufacturer. Joints shall not be placed within 6 feet of the slope face, within 4 feet below top of slope, nor horizontally or vertically adjacent to another joint.

The geogrid reinforcement shall be installed in accordance with the manufacturer's recommendations. The geogrid reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed. The geogrid reinforcement shall be placed in continuous longitudinal strips in the direction of main reinforcement. However, if the Contractor is unable to complete a required length with a single continuous length of geogrid, a joint may be made with the manufacturer's approval. Only one joint per length of geogrid shall be allowed. This joint shall be made for the full width of the strip by using a similar material with similar strength. Joints in geogrid reinforcement shall be pulled and held taut during fill placement.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geogrid reinforcement shall be overlapped or mechanically connected where exposed in a wrap around face system, as applicable.

The Contractor may place only that amount of geogrid reinforcement required for immediately pending work to prevent undue damage. After a layer of geogrid reinforcement has been placed, the next succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geogrid reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geogrid reinforcement and soil. Geogrid reinforcement shall be placed to lay flat and pulled tight prior to backfilling. After a layer of geogrid reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geogrid reinforcement in position until the subsequent soil layer can be placed.

Under no circumstances shall a track-type vehicle be allowed on the geogrid reinforcement before at least 6 inches of soil have been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geogrid reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geosynthetic reinforcement at slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided. During construction, the surface of the fill should be kept approximately horizontal. Geogrid reinforcement shall be placed directly on the compacted horizontal fill surface. Geogrid reinforcements are to be placed as shown on plans, and oriented correctly.

PART III - GEOTEXTILE SOIL REINFORCEMENT

The specific geotextile material and supplier shall be preapproved by ENGEO. The contractor shall submit a manufacturer's certification that the geotextiles supplied meet the respective index criteria set when geotextile was approved by ENGEO, measured in full accordance with specified test methods and standards.

The contractor shall check the geotextile upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the geotextile shall be protected from temperatures greater than 140°F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geotextile will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geotextile damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

Geotextile material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project to assist the Contractor and ENGEO personnel at the start of construction. The geotextile reinforcement shall be installed in accordance with the manufacturer's recommendations. The geotextile reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed, secured with staples, pins, or small piles of backfill, placed without wrinkles, and aligned with the primary strength direction perpendicular to slope contours. Cover geotextile reinforcement with backfill within the same work shift. Place at least 6 inches of backfill on the geotextile reinforcement before operating or driving equipment or vehicles over it, except those used under the conditions specified below for spreading backfill.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geotextile reinforcement shall be overlapped or mechanically connected where exposed in a wraparound face system, as applicable.

The contractor may place only that amount of geotextile reinforcement required for immediately pending work to prevent undue damage. After a layer of geotextile reinforcement has been placed, the succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geotextile reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geotextile reinforcement and soil.

Geotextile reinforcement shall be placed to lay flat and be pulled tight prior to backfilling. After a layer of geotextile reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geotextile reinforcement in position until the subsequent soil layer can be placed. Under no circumstances shall a track-type vehicle be allowed on the geotextile reinforcement before at least six inches of soil has been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geotextile reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geotextile reinforcement as slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided.

During construction, the surface of the fill should be kept approximately horizontal. Geotextile reinforcement shall be placed directly on the compacted horizontal fill surface. Geotextile reinforcements are to be placed within three inches of the design elevations and extend the length as shown on the elevation view unless otherwise directed by ENGEO.

Replace or repair any geotextile reinforcement damaged during construction. Grade and compact backfill to ensure the reinforcement remains taut. Geotextile soil reinforcement must be tested to the required design values using the following ASTM test methods.

TABLE III-1

Geotextile Soil Reinforcements

Property	Test
Elongation at break, percent	ASTM D 4632
Grab breaking load, lb, 1-inch grip (min) in each direction	ASTM D 4632
Wide width tensile strength at 5 percent strain, lb/ft (min)	ASTM D 4595
Wide width tensile strength at ultimate strength, lb/ft (min)	ASTM D 4595
Tear strength, lb (min)	ASTM D 4533
Puncture strength, lb (min)	ASTM D 6241
Permittivity, sec^{-1} (min)	ASTM D 4491
Apparent opening size, inches (max)	ASTM D 4751
Ultraviolet resistance, percent (min) retained grab break load, 500 hours	ASTM D 4355

PART IV - EROSION CONTROL MAT

Work shall consist of furnishing and placing a synthetic erosion control mat and/or degradable erosion control blanket for slope face protection and lining of runoff channels. The specific erosion control material and supplier shall be pre-approved by ENGEO.

The Contractor shall submit a manufacturer's certification that the erosion mat/blanket supplied meets the criteria specified when the material was approved by ENGEO. The manufacturer's certification shall include a submittal package of documented test results that confirm the property values. Jute mesh shall consist of processed natural jute yarns woven into a matrix, and netting shall consist of coconut fiber woven into a matrix. Erosion control blankets shall be made of processed natural fibers that are mechanically, structurally, or chemically bound together to form a continuous matrix that is surrounded by two natural nets.

The Contractor shall check the erosion control material upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the erosion mat shall be protected from temperatures greater than 140°F, mud, dirt, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the erosion mat/blanket shall be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be removed by cutting out a section of the mat. The remaining ends should be overlapped and secured with ground anchors. Any erosion mat/blanket damaged during storage or installation shall be replaced by the Contractor at no additional cost to the Owner.

Erosion control material suppliers shall provide a qualified and experienced representative onsite, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). The erosion control material shall be placed and anchored on a smooth graded, firm surface approved by the Engineer. Anchoring terminal ends of the erosion control material shall be accomplished through use of key trenches. The material in the trenches shall be anchored to the soil on maximum 1½ foot centers. Topsoil, if required by construction drawings, placed over final grade prior to installation of the erosion control material shall be limited to a depth not exceeding 3 inches.

Erosion control material shall be anchored, overlapped, and otherwise constructed to ensure performance until vegetation is well established. Anchors shall be as designated on the construction drawings, with a minimum of 12 inches length, and shall be spaced as designated on the construction drawings, with a maximum spacing of 4 feet.